PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
JANUARY 1956

ORDINARY MEETING

1 November, 1955

DAVID MOWAT WATSON, B.Sc., the retiring President, in the Chair.

The Council recommended to the members present the election as Honorary Members of the Institution: Brigadier-General Sir Harold Hartley, K.C.V.O., M.C., M.A., D.C.L., LL.D., D.Sc., F.R.S., in recognition of his long and distinguished public services; and Professor Sir Geoffrey Taylor, F.R.S., in recognition of his eminence as an applied mathematician and of the importance of his research work in its application to all branches of physical science.

The Council's recommendation was adopted and the two above-named gentlemen were elected as Honorary Members.

The Retiring President said that owing to a change in the method of making awards, such awards would not, in future, be made at the Opening Meeting of the Session. Medals and Certificates would be presented to Authors early in the following year.

The Council reported that they had recently transferred to the class of

Members

Baker, Hugh Thomas, M.A., M.A.I. (*Dublin*).
Baldwin, Marshall Walker, B.Sc. (Eng.) (*London*).
Barnes, Edwin Mayhew, B.Sc.(Eng.) (*London*).
Barratt, John.
Bentham, Max, M.Sc. (*Manchester*).
Blackwood, Edward Austin, B.Sc.(Eng.) (*London*).
Blaikley, David James, M.A. (*Cantab.*).
Blyth, Richard Carleton, M.A. (*Cantab.*).
Brodie, Andrew Alexander Scott, B.Sc. (*Edinburgh*).
Brown, Thomas William, B.Sc.(Eng.) (*London*).
Buick, James Wallace.
Buckley, Edward John, B.Sc.(Eng.) (*London*).
Carne, John Farrant.
Carroll, Basil Patrick, B.Sc.(Eng.) (*London*).
Clancy, Brendan Kenneth, B.Sc. (*Belfast*).
Cullen, Eric Graham, M.B.E., B.Sc. (Eng.) (*London*).
Davidson, Ian, M.Eng. (*Liverpool*).
Dawson, Oliver, B.Sc.(Eng.) (*London*).

Diamond, William Howard, B.A. (*Cantab.*).
Elliott, Charles Rushton, B.Sc. (*Cape Town*).
Golder, Hugh Quintin, D.Eng. (*Liverpool*).
Goodall, John, B.Sc. (*Manchester*).
Green, Leslie Ronald Battman, B.Sc. (Eng.) (*London*).
Greeves, Ivan Sydney Stroulger.
Grey, Ian.
Griggs, Francis Edward, M.A. (*Cantab.*).
Harris, Alan James, B.Sc.(Eng.) (*London*).
Harris, Albert Richard.
Hendry, Professor Arnold William, D.Sc., Ph.D. (*Aberdeen*).
Hill, Arthur William, B.Sc.(Eng.) (*London*).
Hinchliff, Leonard Francis, B.Sc.(Eng.) (*London*).
Hollingshead, James, B.Sc.(Eng.) (*London*).
Humphreys, James Howard.
Hughes, Edwin Patrick Cusack, O.B.E., B.Sc.(Eng.) (*London*).
Johnson, Vernon, O.B.E.
Jones, Henry Frank Harding, M.B.E. M.A. (*Cantab.*).
Kidson, Charles Idrys, B.E. (*New Zealand*).
Kingston, William D'Acre Bennett, B.A.I. (*Dublin*).
Kirkwood, James Watson.
Laing, Percy Lyndon, B.E. (*New Zealand*).
Laing, William Kirby, M.A. (*Cantab.*).
Latto, William, B.Sc. (*Glasgow*).
MacKichan, Ronald William Alexander, D.F.C., B.A. (*Cantab.*).
McNeil, John Struthers, B.Sc. (*Glasgow*).
Matthews, Ian Stanley Gordon, B.A. (*Oxon.*).
Mitchell, Robert, B.Sc. (*Durham*).
Moon, William Edward Penlygon, B.A. (*Cantab.*).
Morice, Seymour Fortescue.
Morley, George William, M.A. (*Cantab.*).
Mowbray, Professor Neil Allman, B.E. (*New Zealand*).
Mulligan, Edward Joseph Butler, B.Sc. (*Edinburgh*).
Murphy, Michael Patrick, B.E. (*National*).
Nougerèdè, Charles Edward de la.
Ollier, Linley Barrett, B.Sc.Tech. (*Manchester*).
Richards, Thomas George, O.B.E.
Rennie, James Warnock.
Rhodes, Malcolm Crosby, B.Sc. (*Birmingham*).
Robertson, Alasdair Ian George Sutherland, M.Sc.(Eng.) (*London*).
Rock, Lionel Gordon Baynes, B.Sc. (Eng.) (*London*).
Smith, William Alexander, B.Sc. (*St. Andrews*).
Somerville, Allan Leslie, M.Eng. (*Sheffield*).
Stayman, Thomas Henry.
Steel, Archibald Mervyn, O.B.E., B.Sc. (*South Africa*).
Sutton, John Richard.
Taylor, James Ingram, M.B.E., B.Sc. (*Aberdeen*).
Thomson, William Risk.
Unitt, John Leslie.
Wallhouse, Harold.
Watson, Ivor Gerald.
Watson, Walter William.
Wilson, James Norman, B.Sc.(Eng.) (*London*).
Yarnold, Douglas Henry.

and had admitted as

Graduates

Aird, Hugh McBeath, Stud.I.C.E.
Aitken, Robert Corbett, Stud.I.C.E.
Anderson, John Michael Douglas, Stud.I.C.E.
Annells, Michael, Stud.I.C.E.
Annison, Peter Faithful, Stud.I.C.E.
Atherton, Geoffrey, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Atkinson, Frank Harry, B.Sc. (*Nottingham*), Stud.I.C.E.
Attard, Joseph, B.Sc.(Eng.) (*London*).
Axell, James Philip, Stud. I.C.E.
Azhar, Mohamed Hamid Ahamed, B.Sc. (*Ceylon*).
Bakker, Jaap Jelle, B.Sc. (*Glasgow*), Stud.I.C.E.
Bancroft, Michael John, B.Sc. (*Cape Town*).
Bashford, John Frederick, B.Sc. (Eng.) (*London*).
Batty, Robert Broomhead, Stud.I.C.E.
Beard, Roy Sinclair, Stud.I.C.E.
Beaton, Harold Campbell, B.Sc. (*Glasgow*), Stud.I.C.E.

Beer, Colin Robert, Stud.I.C.E.
Benham, David Lee, B.A. (*Oxon*), Stud. I.C.E.
Betteley, John Dennis, B.Sc. (*Wales*), Stud.I.C.E.
Betterton, Peter Raymond, B.Sc. (*Wales*).
Beusmans, Derrick Matthew, Stud.I.C.E.
Beveridge, Andrew, B.Sc. (*Glasgow*). Stud.I.C.E.
Billett, Albert Clive Kilvert, B.Sc. (*St. Andrews*).
Bisogno, Sabino Vittore, B.Sc. (*Cape Town*).
Blacker, George Allan, Stud.I.C.E.
Blair, John, B.Sc.Tech. (*Manchester*).
Bousfield, Christopher Bird, Stud. I.C.E.
Bowering, Robert Henry, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Bowen, Richard Alexander, B.E. (*National*).
Bracegirdle, Ronald Bernard, B.Sc. Tech. (*Manchester*), Stud.I.C.E.
Bradbury, John Norman, B.Sc. (*Birmingham*), Stud.I.C.E.
Bray, Peter Henry, B.A. (*Cantab.*), Stud.I.C.E.
Bridges, Brian Crawford, B.Sc. (*Leeds*), Stud.I.C.E.
Brisley, Eric Geoffrey, Stud.I.C.E.
Broomfield, Brian John, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Brown, William Trevor Dennis, B.Sc. (Eng.) (*London*).
Callard, Anthony John, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Carroll, Leo Joseph, B.Eng. (*Liverpool*), Stud.I.C.E.
Cato, Bruce Hillier, B.E. (*New Zealand*).
Chan, William Wai-Lee, Ph.D. (*London*), B.Sc. (*Hong Kong*), Stud.I.C.E.
Challenor, Dennis, Stud.I.C.E.
Clark, William Alexander, Stud.I.C.E.
Cochrane, Graham Hugh, B.A. (*Cantab.*) Stud.I.C.E.
Coleman, Brian Edward, Stud.I.C.E.
Cook, Roy John Leonard, B.Sc.(Eng.) (*London*).
Cox, David Dingley, Stud.I.C.E.
Crawford, Duncan McCallum, Stud. I.C.E.
Crawford, John Cran, B.Sc. (*Glasgow*), Stud.I.C.E.
Cuthbertson, Alexander, Stud.I.C.E.
Dangerfield, Leslie Beresford, Stud. I.C.E.
Davies, Barry Fraser, B.A., B.A.I. (*Dublin*).
Devereux, Walter Frank, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Dias, Alahapperumaarachchige Dharmaransi, B.Sc.(Eng.) (*London*), Stud. I.C.E.
Dove, Jack Ernest, Stud.I.C.E.
Dowell, James Hendry, B.E. (*New Zealand*).
Duffy, Christopher Thomas, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Duncan, Michael Anthony Robert Edward Grant, B.Sc.(Eng.) (*London*).
Edmonds, David Thomas, B.Sc. (*Leeds*), Stud.I.C.E.
Elkington, John, Stud.I.C.E.
Elliott, Robert Willoughby, Stud. I.C.E.
Evison, John, B.Sc. (*Manchester*), Stud. I.C.E.
Eyles, David Roadknight.
Fennell, Harold Cecil, Stud.I.C.E.
Fisher, David Robin, B.A. (*Cantab.*).
Fletcher, Richard John, B.Sc.(Eng.) (*London*).
Forber, Thomas Keith, B.Eng. (*Liverpool*).
Foulkes, Robert Ogden, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Fraser, Donald Edward Kerr, B.Sc. (*Aberdeen*).
Frederick, Melville Radcliffe, B.Sc. (*Nottingham*), Stud.I.C.E.
Fuller, John David Cresswell, Stud. I.C.E.
Fussell, David Richard, Stud.I.C.E.
Gammie, Donald, Stud.I.C.E.
Ganguly, Amal Kumer, B.Sc. (*St. Andrews*).
Gee, Brian Lewis, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Gilchrist, Alexander, Stud.I.C.E.
Gillies, Alasdair, B.Sc. (*Glasgow*).
Goonewardene, Ananda Amarasiri, Stud.I.C.E.
Greensides, Jack, B.Eng. (*Liverpool*), Stud.I.C.E.
Guha, Sukumar, Stud.I.C.E.
Haines, Peter James Terry, Stud.I.C.E.
Hale, Peter William, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Handa, Virender Kumar, B.Sc.(Eng.) (*London*).
Harborne, Peter Frank, B.Sc. (*Bristol*).
Harper, Anthony Stuart, Stud.I.C.E.
Harrington, John Frederick, B.Sc. Tech. (*Manchester*), Stud.I.C.E.
Harris, George Graham, B.Sc.(Eng.) (*London*) Stud.I.C.E.
Harrison, Robert Anthony, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Harvey, Ian Sherriff, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Herrick, David William, Stud.I.C.E.

Hilson, Barry Oliver, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hoal, Gerald Raymund, B.Sc. (*Witwatersrand*).
Hobbs, John, Stud.I.C.E.
Hole, Leslie Elliot, Stud.I.C.E.
Holmes, John Anthony Gardner, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Holland, Derek Ronald Edward, B.Sc. (*Glasgow*), Stud.I.C.E.
Holt, John Kendrick, B.Sc. (*Glasgow*), Stud.I.C.E.
Hooi Kah Hung, B.C.E. (*Melbourne*).
Horn, Geoffrey Frederick, Stud.I.C.E.
Horner, Michael John, Stud.I.C.E.
Horsey, Ronald John, Stud.I.C.E.
Hubbard, Raymond, Stud.I.C.E.
Hudson, Joseph William, Stud.I.C.E.
Hughes, Colin, Stud.I.C.E.
Humpidge, Henry Barry, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Ingham, Bryan Ronald, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Irvine, James, B.Sc. (*Glasgow*), Stud. I.C.E.
Jackson, Brian James, B.E. (*New Zealand*), Stud.I.C.E.
Jacques, Gordon, B.E. (*Tasmania*).
Jansz, Edgar Rupert, B.Sc. (*Ceylon*).
Jennings, Anthony Joseph, B.A., B.A.I. (*Dublin*).
Jones, Francis Clifford, B.Eng. (*Sheffield*), Stud.I.C.E.
Jordanou, Michael Georgiou, B.Sc. (Eng.) (*London*).
Johnson, Stephen Richard Hall, B.A. (*Cantab*).
Kay, John Lazenby, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Kaye, John Cooper, B.Sc. (*Leeds*), Stud.I.C.E.
Keedwell, Michael James, B.Sc. (*Manchester*), Stud.I.C.E.
Kermode, Brian Charles Davis, B.C.E. (*Melbourne*).
Kershaw, John Brian, B.Eng. (*Liverpool*).
Kottegoda, Nathabandhu Thilaksri, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Kyobe, Wilson, B.Sc. (*Wales*).
Langan, Donald, B.Sc.Tech. (*Manchester*).
Langdon, David, B.Sc.(Eng.) (*London*).
Law, Keith Edward, B.Sc. (*Nottingham*), Stud.I.C.E.
Leary, Russell Charles, B.E. (*New Zealand*).
Le Clerc, Brian Richardson, Stud.I.C.E.
Lee, Anthony Joseph, B.A., B.A.I. (*Dublin*).
Leighton, Neil Harry, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Leong Yoke Sun, B.Sc. (*Wales*).
Leveridge, Peter James, Stud.I.C.E.
Light, Peter, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Lloyd, Michael John, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Lucas, Colin John, B.Sc.(Eng.) (*London*)
Lucas, Edward Thomas Paschal, B.E. (*National*).
Luchford, Harry John Moore, Stud. I.C.E.
Lugg, Herbert Kenneth Michael, Stud. I.C.E.
McCabe, Eugene Bernard, B.E. (*National*).
McGwynne, Denis Edward, B.E. (*National*).
McInnes, James Aitken, Stud.I.C.E.
McKay, Robert Alexander.
Mackie, Alexander, Stud.I.C.E.
Madden, Patrick Gerard Joseph, B.E. (*National*).
Marriott, Keith Howard, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Marshall, John, B.Sc. (*Cape Town*).
Mayfield, Brian, B.Sc. (*Nottingham*), Stud.I.C.E.
Metcalfe, David Hugh, B.Sc. (*Leeds*), Stud.I.C.E.
Metherell, Robert Colin.
Michell, Christopher John, B.Sc. (*Bristol*), Stud.I.C.E.
Miles, Alan David George, Stud.I.C.E.
Milne, James Sangster.
Moore, Robert Hollingsworth, Stud. I.C.E.
Morris, Angus Robin, B.Sc. (*Witwatersrand*).
Moses, Peter, B.Sc.(Eng.) (*London*) Stud.I.C.E.
Munns, Thomas Trevor, Stud.I.C.E.
Murphy, Eugene Anthony, B.Sc. (*Belfast*).
Nancarrow, David Rodney, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Naylor, David John, B.A. (*Cantab.*).
Neill, Thomas William Robertson Stud.I.C.E.
Newman, Paul Brendan, Stud.I.C.E.
Nisbet, Richard Frederick, B.Sc (*Nottingham*), Stud.I.C.E.
Nunoo-Quarcoo, Daniel Akai, B.Sc (*Glasgow*).
Nutt, Edward Daniel, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Osborne, George Barry, Stud.I.C.E.
Olley, Peter Michael Rex.
Olugbekan, Olusoji, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Opalinski, Aleksander, B.Sc.(Eng.) (*London*).
Palmer, Denis Norman.
Pancholi, Vijayshanker Ravishanker Stud.I.C.E.

Parkinson, John Blythe, Stud.I.C.E.
Paterson, Daniel Griffiths, Stud.I.C.E.
Patience, Derek Frank, B.Sc. (*Leeds*), Stud.I.C.E.
Pattie, William, Stud.I.C.E.
Paul, Robert Menzies, B.Sc. (*Edinburgh*), Stud.I.C.E.
Payne, Philip George, Stud.I.C.E.
Pegler, Michael Richard Holmes, B.Sc. (*Witwatersrand*).
Perry, Edwin John, B.E. (*New Zealand*).
Perera, Edmund Fedrick Marcus, B.Sc. (*Ceylon*), Stud.I.C.E.
Pettit, James Jethro, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Phoenix, John Richard, B.A., B.A.I. (*Dublin*).
Pike, John David Lockwood, Stud.I.C.E.
Pope, Geoffrey Arthur, Stud.I.C.E.
Porter, John Reid, Stud.I.C.E.
Potter, Uter Alan, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Poulton, Charles George, B.Sc. (*Witwatersrand*).
Powell, Daniel Price, B.Sc. (*Wales*).
Powell, Dennis Vivian, B.Sc. (*Wales*), Stud.I.C.E.
Raikes, Thomas Douglas, B.A. (*Oxon*).
Rehani, Adel.
Rich, Michael Mervyn, Stud.I.C.E.
Ridley, Tony Melville, B.Sc. (*Durham*), Stud.I.C.E.
Roberts, William Charles, B.Sc.(Eng.) (*London*).
Robinson, William Whitfield, Stud. I.C.E.
Rocheford, Maurice Alva, B.Sc. (*Glasgow*), Stud.I.C.E.
Ross, Peter.
Ross, William McMillan.
Rynkiewicz, Leonard Stanislaw, B.Sc. (Eng.) (*Natal*), Stud.I.C.E.
Sankey, David Augustine, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Saunders, Charles Robert, B.Sc. (*Glasgow*), Stud.I.C.E.
Sayer, Colin Francis, B.Sc.(Eng.) (*London*).
Seale, Charles O'Malley Haskins, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
Selfe, Michael Aylwin, Stud.I.C.E.
Seneviratne, Abeyaratne Bandara Ehelepola, Stud.I.C.E.
Shanks, Peter Henry Francis, B.A., B.A.I. (*Dublin*).
Sharkey, Liam Gerard, B.Sc. (*Belfast*).
Shaw, James Maxwell, Stud.I.C.E.
Shaw, Kenneth, Stud.I.C.E.
Sheppard, Edward Arthur Robin, Stud.I.C.E.
Sherratt, Reginald Henry, B.Eng. (*Liverpool*).
Shields, David Brian, B.Sc. (*Belfast*), Stud.I.C.E.
Shields, David Hunter, B.Sc. (*Glasgow*), Stud.I.C.E.
Sides, Geoffrey Raymond, B.Sc. (*Manchester*), Stud.I.C.E.
Simcock, Michael Ainslie, B.Sc.Tech. (*Manchester*).
Sims, Frank Alexander, B.Sc. (*Nottingham*).
Smith, Derek Robin Stewart, Stud. I.C.E.
Smith, John Geoffrey Saville, B.Sc. (*Glasgow*).
Smith, Maven, Stud.I.C.E.
Snaith, Kenneth Ivan, B.Sc. (*Durham*).
Snowball, David John, B.Sc.(Eng.) (*London*).
Stares, Arthur Robert, B.E. (*Queensland*).
Steedman, William Ronald.
Stephens, Clayton Rayner Wilfred, Stud.I.C.E.
Straw, Andrew James, B.C.E. (*Melbourne*).
Strouts, Edward Robert, B.Sc.(Eng.) (*London*).
Sulley, William David, B.Eng. (*Liverpool*).
Tait, Ian Campbell, B.Sc. (*Natal*).
Tanton, Malcolm Stanley, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Taylor, Charles William Derek, B.Sc. (*Durham*).
Taylor, Patrick Hamilton, B.Sc. (*Aberdeen*).
Thomas, Jeffrey William, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Thomas, Roger Wilson, B.Sc.Tech. (*Manchester*).
Thompson, Colin Frederick Richard, Stud.I.C.E.
Thomson, Alexander Keith, Stud.I.C.E.
Thomson, James Crawford, Stud.I.C.E.
Thomson, John.
Thomson, John Stuart, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Turnbull, John Chilton, Stud.I.C.E.
Turner, Leonard Arthur, B.Sc.(Eng.) (*London*).
Usher, Peter Malcolm, B.A. (*Cantab.*), Stud.I.C.E.
Vyvyan, Bernard Jeremy, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Walker, Brian John, Stud.I.C.E.
Walker, Geoffrey, Stud.I.C.E.
Walker, Gerald Boyd, Stud.I.C.E.
Walker, Peter Francis, Stud.I.C.E.
Warren, Peter, B.Sc. (*Bristol*).
Watermeyer, Christopher Frederick, B.Sc. (*Cape Town*).

WATERSON, WILLIAM MALYON, B.Sc.(Eng.) (*London*), Stud.I.C.E.
WHITTAKER, RODERICK THOMAS, Stud. I.C.E.
WEEKS, ALAN GEORGE, Stud.I.C.E.
WIJESUNDERA, SENARATH DE SILVA, B.Sc. (Eng.) (*London*), Stud.I.C.E.
WILDE, PETER MALCOLM, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
WILKES, PETER FRANCIS, B.Sc. (*Birmingham*), Stud.I.C.E.
WILLETT, MICHAEL JOHN FARQUHAR, Stud.I.C.E.
WILLIAMS, JOHN KENNEDY, Stud.I.C.E.
WILSON, ROBIN LEE, B.Sc. (*Glasgow*), Stud.I.C.E.
WINDLE, GEOFFREY VINCENT, B.Sc.Tech. (*Manchester*).
WOLLEY, CHARLES WILLIAM, Stud.I.C.E.
YAP CHIN LEONG, B.Sc. (*Bristol*), Stud. I.C.E.

and had admitted as

Students

ABRAHAM, ALUN GARETH.
AKESTER, MICHAEL GORDON.
AKPAN, ETIM ROBINSON.
ALLAN, GEORGE OLIVER.
ALLISON, MICHAEL.
ANDERSON, PERCY GEORGE.
ANDREWS, STANLEY ERNEST ROBERT.
ASPINALL, HAROLD DAVID.
BANKS, JAMES ACFIELD.
BARTLETT, DEREK ALBERT CYRIL.
BAYNES, MICHAEL.
BAYS, ALEXANDER HENRY.
BEACH, JOHN MARSHALL.
BEASANT, ROBERT HENRY.
BELL, EDWARD ARNOLD.
BELL, RONALD AUBREY.
BENJAMIN, BRIAN.
BERRY, GEOFFREY PETER.
BERNET, MICHAEL MEIR MANFRED.
BINDING, WERNER AUGUST.
BLENKHORN, GEORGE HALLAS.
BLOWER, ALAN FRANK.
BOUGH, REGINALD PETER.
BRACKEN, NEVIL KINGSLEY.
BRADLEY, EDWARD MAXWELL.
BRAYBROOKS, JOHN.
BREWSTER, PETER.
BROOKER, MICHAEL JOHN.
BROWN, DAVID.
BROWN, WILLIAM LINDSAY.
BRYCE, SEAN.
BURGESS, PETER JOHN OAKLEY.
BURKE, BRIAN READE.
CAMPBELL, NEIL DOUGLAS MITCHELL.
CAMPBELL, TERENCE GEORGE.
CANTRELL, ALAN CHARLES.
CARBY, MICHAEL RANDALL.
CHARLES-JONES, WILMOT SELWYN.
CHAPMAN, MALCOLM CAMPBELL.
CHRIMES, LAURENCE JOHN.
CLARKE, PETER.
COATES, JOHN MERYON.
COLLINSON, THOMAS ANTHONY.
CONNOR, PETER.
COOK, BRIAN JOHN.
COOKE, JOHN CHRISTOPHER.
COOPER, JOHN BARRY.
COUSINS, ALAN DAVID.
COZENS, DAVID GERALD.
CROW, PATRICK LEWIS.
CRUDDAS, EDWARD ALAN.
DALTON, RAYMOND CHARLES RUSSELL.
DAVIES, GILBERT.
DAVIES, JOHN ARNOLD.
DAWSON, CHARLES DAMAR.
DAWSON, RONALD HENRY.
DEAKIN, PETER.
DIXON, RAYMOND ERNEST.
DOOGAN, ERIC CHARLES.
DOWNER, JOHN SINCLAIR.
DUFFELL, JAMES ROGER.
DUNMORE, MICHAEL HARVEY.
DURRANT, WILLIAM JAMES.
EAREY, MICHAEL JOHN.
EMUCHAY, CLEMENT OBOADA.
FICK, BENJAMIN ALEXANDER.
FOO CHEE JIN.
FOSTER, KENNETH DENNING.
FROST, ANTHONY WOOD.
FULLER, DAVID HORACE ROBINS.
GAN CHING LIM.
GARDEN, ROBERT LEWIS.
GAUTREY, BRIAN WILLIAM.
GOUDGE, ALAN EDWARD.
GOURLAY, MICHAEL RICHARD.
GRANT, MICHAEL ANTHONY.
GREGG, ROBERT JOHNSTON.
GREIG, COLIN LANGLEY.
GRIMSTON, REGINALD.
GWYTHER, MICHAEL WILLIAM.
HAMP, RICHARD ANTHONY.
HAMPSON, WILLIAM ROWLAND.
HANCOCK, LESLIE RICHARD.
HARRIS, KENNETH ARTHUR.
HARRISON, DONALD.
HARRISON, FREDERICK.
HARRISON, KENNETH PETER.
HARSTON, JOHN EDWARD.
HEIGHTON, ANTHONY TREVOR.
HENDERSON, BRUCE.

Hill, Derek Richard Rowland.
Hill, Edward Ian.
Hills, Peter.
Hollanders, Kenneth.
Hook, David Lance.
Hooper, Brian Neil.
Howden, Carl.
Hughes, David Edward Blair.
Husain, Abdul Khalik Abdul.
Hutchings, Robert Leonard.
Jeffries, David John.
Jenkins, Gareth Stewart.
Jones, John Arthur.
Kain, Eric Samuel.
King, Dennis Stewart.
Kinghorne, James Phillip.
Kisch, Michael Samuel.
Knight, David Raymond.
Knights, Terence Arthur.
Lack, Anthony Arthur.
Law, Allan Gordon Graham.
Lean, John Leonard.
Leahy, John Michael Hau.
Lendon, Barry John.
Lewis, John Noel.
Lovely, Douglas Leonard.
McCullagh, Edward McKim Lyell.
McDougall, Roderick.
Macfarlane, Robert David.
Mackelden, Bryan Edmund.
Mackereth, George Michael.
McKerral, John Morrison.
Mackintosh, Neil Cameron.
McLennan, Gordon.
McMillan, William Anthony.
McRory, Robert.
McSporran, Ian Greer.
Maddaford, William.
Mahy, Bertrand Lewis.
Malhotra, Virindar Mohanlal.
Mathews, Roger George.
Matthews, William Graham.
Meagher, Richard John.
Mendis, Warnaculasuriya Wadumestrige Hugh Emmanuel.
Millar, John.
Milliard, Russell John.
Miller, Alexander William.
Mills, Brian Ivor.
Moore, Edward.
Moren-Brown, David Michael.
Mordecai, Victor Solomon.
Morrison, Brian Alexander.
Moses, John Charles.
Mosson, John Geoffrey.
Mudd, Donald Norman.
Mulley, Anthony Cyril Conroy.
Naylor, David.
Newton, Calvert John.
Newton, John.
North, William David.
Nuttall, John Hedley.
O'Byrne, Christopher.
Okeke, Gilbert Asombanya.
O'Mer, Robin.
Organ, Nigel Stephen.
Osborne, John Bernard.
Owsianka, Adam Andrzey.
Park, Robert.
Parker, Stanley.
Parkes, Anthony George David.
Patrinos, Nicholas.
Parsons, Bryan Charles.
Paynter, Edward William.
Peacock, William James.
Penn, Richard David.
Perera, Gallara Acharige Abeysiri.
Phillips, David Brian.
Phillips, Frank Robert.
Proud, Robert Gilmour.
Queen, Charles Geoffrey.
Ranatunga, Semba Kuttige.
Ranasinghe, Upali.
Reading, Bernard Barnes.
Reeves, John.
Reid, John Blyth.
Richards, Derek James.
Richardson, Alan.
Riley, John Kenneth.
Roberts, Eryl Wyn.
Robson, Keith.
Robinson, Christopher.
Rochester, Terence Anthony.
Rock, George.
Rowntree, John Pickering.
Ruscoe, Ian Hedley.
Sanmuganathan, Karthigesu.
Sanyal, Kalyan Kumar.
Saunders, Robert Leslie.
Scott, John Leonard Maurice.
Seller, Brian.
Sexton, Christopher John.
Shalders, Michael Bloy.
Sharland, Derek Charles.
Shaw, Ian Christopher.
Shaw, Keith Charles.
Sheldon, William Gordon.
Simmons, Jeffrey Charles Frederick.
Smith, Alwyn Aubrey.
Smith, Ian Kidd.
Spencer, Geoffrey.
Spry-Bailey, Philip.
Story, Ernest Graham.
Steele, Francis.
Sturt, Andrew Neil.
Styman, Michael John.
Sutton, David John.
Tai Kon Chin.
Taylor, John Colin.
Thomson, William Davidson.
Timms, John Reginald.
Timmington, Geoffrey Wilfred.
Tolner, Victor Henry.
Tory, Arthur Colin.

Tye, Peter Frederick.
Vearncombe, Alan.
Verwaard, Jan Marinus.
Wahid, Abdul Chowdhry.
Walker, Derek.
Walker, Michael Kilby.
Warner, Michael John.
Walters, Hugh Colin.
Weatherhead, Owen Victor.
Webber, Edward Sutton.
Weightman, Brian.
West, Ronald Villiers.
Whiting, John Gilbert.
Wilkes, Peter Francis.
Wilson, Andrew John.
Winspear, David Stanley.
Wrightson, Barry Lyndon.
Wylie, Harold Alan.
Yarwood, Colin.
Ying Yok Hang.

The Retiring President, introducing the new President, said that Mr Wallace had served on the Council of the Institution for eleven years, and during that time he had in all probability been a member of all the Standing Committees and of a great many of the *ad hoc* Committees. He had also done an enormous amount of work for the Institution in other ways. He had represented it, for instance, at the Second International Conference on Soil Mechanics and Foundation Engineering at Rotterdam in 1948, and on the National Consultative Council of the Ministry of Works, the Engineering Divisional Council of the British Standards Institution, the Executive Committee of the Building Research Congress, 1951, the British National Committee of the International Society of Soil Mechanics and Foundation Engineering, the General Board of the National Physical Laboratory, the Council for Codes of Practice for Buildings, the Civil Engineering Codes of Practice Joint Committee, and the Engineering Joint Council, and he had been Chairman of the Mining and Subsidence Committee. Quite apart from all that, Mr Wallace had had a very distinguished professional career.

During that time all the members of Council had learnt to know and to appreciate his sterling qualities—and that he had the welfare of the Institution very much at heart.

Mr William Kelly Wallace, C.B.E., then took the Chair and called on Professor A. J. S. Pippard to move a resolution.

Professor A. J. S. Pippard, Vice-President, moved the following resolution:

> "That the members present at this Meeting desire, on behalf of themselves and others, to record their high appreciation of the services rendered to the Institution by Mr D. M. Watson during his term of office as President."

He said that on the same day, twenty years ago, a distinguished civil engineer had taken the Chair as the new President of the Institution, and in his Presidential Address he had made the following remark: "It is our duty to help our sons to do greater and better things than their fathers." A year ago an event had taken place which Professor Pippard believed to be unique in the annals of the Institution; the son of that President of twenty years ago had taken office in his turn. During the past year all members of the Institution had had the benefit of the great devotion and selfless service of Mr Watson, and he had proved without any doubt that his father, at any rate, had lived up to his precepts.

The large number of Corporate Members, Students, and Graduates who had met Mr Watson in the course of this year's work had all been struck by his unfailing friendliness—that was his chief characteristic—but it had probably remained for those who had had the honour to serve under him on the Council to realize to the full how devotedly he had given himself to the service of the Institution.

It was therefore, said Professor Pippard, his very great privilege and happiness to propose the resolution. All the members of Council would look back upon the past year with gratitude to Mr Watson for his inspiration.

Mr William Linn, seconding the motion, said that Mr Watson must surely have felt the warmth of the feeling of all the Members towards him during the past year.

The Institution was very old and it was steeped in a tradition of grand men at its head; it was the opinion of the members present that the Institution was the richer for its year under Mr Watson's guiding hand.

Mr Watson, as had been said, had not spared himself during his Presidency. Members in the provinces knew how welcome he had been on his travels when he had come among them at their various functions and had been very struck with how he mingled a sincere friendliness with the upholding of the dignity of his high office. It was a very great physical strain travelling all over the country and attending such functions but he did not fail even when extra calls were made upon him from far-away branches.

In the new List of Members there was a long list of Past-Presidents of the Institution. They were grand names, great names, associated with the growth of the Institution and its glorious past. The name of David Watson was about to be added to that List. Mr Linn was confident that the sons of today looking back in some future List to the year 1954–55 would say—"Those were great days!"

The motion was carried by acclamation.

The President then delivered his Address.

PRESIDENTIAL ADDRESS OF

William Kelly Wallace, C.B.E.

President, 1955–56

To become President of the Institution is the highest honour to which a Member can aspire, and I desire to express my appreciation of the distinction you have conferred by electing me to this office. I shall endeavour to carry out the many duties of the position to the best of my ability and hope to serve the Members, the Institution, and the profession in a manner which to some extent will justify your trust in me.

During my professional working life I have been a Railway Civil Engineer. Two eminent Railway Engineers have been President in the recent past, one in 1949 and the other in 1952, so it is obvious that the railway field, though fertile, has been under rather intensive cultivation. Consequently, I decided to follow Mr Quartermaine's example and bring to your notice some of the early days of railway construction, particularly that of the first railway designed and constructed to be a trunk line, namely, the London and Birmingham.

The first Bill, promoted in 1832, had intended the London terminus to be at or near King's Cross, but was rejected by the House of Lords. In the next Bill, promoted and passed in the following year, the London terminus was at Camden Town.

The prospectus issued by the promoters in 1830 shows the engineers as Messrs Stephenson & Son, but George Stephenson left the design and laying-out of the line to his son Robert, though he was probably consulted frequently.

On the 13th September, 1833, the Directors appointed Robert Stephenson, Engineer-in-Chief, and a covenant and agreement was signed on the 21st September, 1833, by Robert Stephenson and six of the Directors of the London and Birmingham Railway. Under its terms, the engineer "shall devote the whole of his time and talents in laying down the said line of railway from London to Birmingham superintending the execution thereof and in the performance of all the duties which devolve upon a civil engineer employed and acting under similar circumstances. He shall not be engaged in any other occupation and he shall not absent himself from but shall permanently reside on or near the line" . . . "For the above-mentioned services the said Robert Stephenson shall be paid by the said Company a salary of £1,500 per annum from the date of his appointment . . . and for travelling and all other contingent expenses the further sum of £200 per annum, such salary and allowance to be paid quarterly."

I thought it would be of interest to find out if possible what the equivalent of this salary would be today and consulted a bank official. He kindly went to some trouble in the matter and transmitted the following information The only index numbers linking 1834 and 1954 are for wholesale prices. On the basis of these, £1,500 in 1834 seems to have had the same purchasing power of about £4,000 today. This figure cannot be accepted as the equivalent on the basis of the value of a person's income. The pattern of spending has changed and many items which are common, or even essential, today were unobtainable in 1834. In addition, there is the question of income tax and surtax which did not exist in 1834. Allowing for tax, about £12,000 a year is required today to yield a net £4,000. Perhaps the most one can say is that, allowing for tax, £1,500 a year in 1834 is equivalent to something of the order of £12,000 today.

The length of the authorized line was 111¼ miles and the maximum gradient 1 in 330, or 16 ft/mile. The total length of gradients exceeding 14 ft/mile, or 1 in 377, was 25½ miles ascending from London and 18½ miles descending from London. The total length of "Level Planes" was 9¾ miles. The line was double throughout, with passing loops at intervals.

A general meeting of Proprietors was held on the 19th September, 1833. The passing of the Bill in the last Session of Parliament was reported, twenty-four Directors appointed, and, doubtless owing to the slow means of communication then available, the line was divided into two portions under the separate superintendence of committees of the Directors meeting in London and Birmingham respectively. Stephenson reported to both committees direct, and at intervals to the Board as a whole.

At the fifth half-yearly meeting on the 18th February, 1836, the whole line, including the Euston extension, was under contract, but a quicksand had been found in the tunnel at Kilsby which was to be a source of great trouble, expense, and delay.

At the seventh half-yearly meeting on the 3rd February, 1837, the shareholders were told that "the entrance to the London passenger station opening immediately upon what will necessarily become the grand avenue for travelling between the metropolis and the midlands and northern parts of the kingdom, the Directors thought that it should receive some architectural embellishment. They adopted accordingly a design by Mr Hardwick for a grand but simple portico which they consider well adapted to the national character of the undertaking." The reaction of the shareholders is unknown; the cost was £35,000.

The line was opened in sections, due to delays in completing some of the works. Euston to Boxmoor, 24½ miles, on the 20th July, 1837; Boxmoor to Tring, 7 miles, on the 16th October, 1837; Tring to Denbigh Hall, 16½ miles, on the 9th April, 1838, and Birmingham to Rugby, 29 miles, on the 9th April, 1838. Passengers were carried by road between Denbigh Hall and Rugby until the whole line from London to Birmingham, 112¼ miles, was opened on the 17th September, 1838.

Permanent way

Stephenson, in a letter to the Secretary of the Birmingham Committee, Captain Moorsom, R.N., on the 8th December, 1834, recommended for the permanent way fish-bellied rails 50 lb/yd on chairs of 20 to 22 lb. each "with a fastening that allows of some deflexion," supported on stone blocks not less than 2 ft square by 1 ft thick in cuttings, with sleepers of larch or oak not less than 10 in. × 6 in. × 8 ft 6 in. long on embankments. For fastenings, oak trenails and spikes for fixing chairs to the blocks, and spikes only on sleepers.

At the third half-yearly meeting of shareholders on the 13th February, 1835, the Chairman said "impressed with the importance of ensuring the adoption of sound principles in the construction of Railway Bars and Supports, the Directors have endeavoured to collect the best information on the subject from scientific and practical men. In furtherance of this object, they have decided upon undertaking experiments on malleable iron bars of different forms at the suggestion and under the direction of Professor Barlow at Woolwich Dockyard, and they doubt not that the result by supplying Data where hitherto there has only been unsatisfactory theory or imperfect experience, will be advantageous not only to this Company but to railways in general."

At the next half-yearly meeting on the 7th August, 1835, the shareholders were

told that Professor Barlow had reported "throwing considerable light on that important question." Further details of the "light" are unavailable, but perhaps he advised a parallel section of greater weight, since at the meeting on the 18th February, 1836, we are told the Directors "have decided to use rails 75 lb/yd upon bearings 5 ft apart with the exception of the 20 miles near London which will be laid with rails 65 lb/yd on bearings 4 ft apart."

At the thirteenth half-yearly meeting of the Company on the 7th February, 1840, the Directors reported that the maintenance of the permanent way had been let out to contract for almost all the line.

Earthworks

The decision to construct the railway with easy gradients and curves resulted in heavy earthworks. At that time excavation was a manual job, except where aided by explosives in rock. Embankments were formed by end-tipping from formation level or from borrow pits alongside.

Cutting was run to fill, where possible and economical, in end-tip wagons holding about 1 ton, horse-hauled for short leads, and by locomotives for long. Borrow pits appear to have been more used than would be the case today, but that is probably due to relative costs being different. Surplus excavation from cuts was spread alongside the top of the slope. Barrows were hauled up from the formation level by horse-powered gins, and this plant was also used in forming banks from side cutting (Fig. 1, facing p. 16).

There was considerable trouble from settlement and slips. The two items which delayed the opening of the line were Kilsby Tunnel and Blisworth (now Roade) cutting.

On the 17th February, 1837, Stephenson reported that "the unfavourable weather and the slippery nature of the clay lying upon the rock at Blisworth had caused the embankment at Ashton to slip forward and sideways very much. After examining it, I decided upon increasing the side cutting at Ashton and throwing the clay lying upon the rock in the cutting entirely into spoil, as it appears quite unsuitable for forming embankment."

Below the rock referred to above, which was soft and fissured, lay another stratum of clay, the weathering of which would have led to dangerous falls of boulders. To prevent these an "undersetting" of masonry was provided in the form of a slender face wall 20 ft high and tapering from 4 ft to 2 ft thick and strengthened by buttresses at 20-ft centres, tapering from 9 ft 6 in. to 4 ft 6 in. in thickness, and from 6 ft to 4 ft width on face. An arched invert 6 ft wide spanned between the feet of each pair of buttresses.

On the 17th February, 1838, in a general report to both Committees on the time required to finish the line, Stephenson stated "between London and Tring the permanent road is in tolerably good order except the Brent embankment near London and on the Colne embankment near Watford. Both these works have continued to subside with scarcely any intermission, the former from the slippery nature of the material which comprises it, the latter from the unsoundness of its substratum in the valley of the Colne."

Bridges and viaducts

Bridges, though numerous, were not of outstanding span or height, and this also applies to the thirteen viaducts. Construction was usually in red brick, the few

structures in stone were probably so built to meet the requirements of the landowner. Stone copings were usual and stone voussoirs were sometimes employed.

Four underbridges carrying the line over the Grand Junction Canal were built, with cast-iron arches and spandrels on the skew, and varying in span from 45 to 68 ft. The original superstructures are still in use, additional spans to carry the widened line being added later.

A cast-iron tied arch was constructed over the Regent's Canal at the top of the incline from Euston. Minimum construction depth was essential, and was achieved by taking up the arch thrust by wrought-iron tie-rods and providing a cross-girder under each pair of rail chairs. The deck was formed of cast-iron gratings. This bridge carried four roads by means of three half-through tied arches.

Viaducts were built with brick arches, except in one case where lack of headroom necessitated the use of cast-iron girders. The maximum span was 60 ft. The largest of these structures was Wolverton Viaduct carrying the line over the Bedfordshire Ouse. It has six elliptical spans of 60 ft and eight semi-circular arches of 15-ft span.

TUNNELS

Although tunnels had been constructed for canals—some of considerable length—a number of engineers and a large proportion of the general public considered them undesirable, if not dangerous on railways. Not for structural reasons, but from fears that the atmosphere in the tunnel would become so foul from the passage of locomotives as to cause illness, if not death, to the passengers and staff in the trains.

Stephenson obviously held a different opinion, since the line called for eight tunnels varying in length from 288 yd to 1 mile 666 yd. All were built for double line, and the span seems to have varied from 22 ft to 23 ft 6 in. Construction was generally of brickwork, with stone portals in some cases.

There must have been considerable apprehension expressed about the tunnels, as the Board of the 6th August, 1835 minuted "that the Engineer in Chief be directed to report on the expediency of constructing air shafts 40 ft long by 30 ft wide or thereabouts in all tunnels on the line, in such places as may be found most convenient, so that the space between the air shafts shall be about 600 yd and not further from each end than 600 yd, and in case the Engineer should advise the same to be constructed, that the respective Committees (Birmingham and London) be requested to make the necessary arrangements for causing them to be forthwith proceeded with."

As no further mention is made of this proposal, it would appear that the report of the Engineer-in-Chief was averse to their construction.

It is recorded that after the completion of Primrose Hill Tunnel (1,154 yd), a special train conveying engineers and doctors was run into the tunnel, stopped for 20 min while the locomotive blew off steam and fouled the atmosphere generally, then ran clear of the tunnel and finally backed through to the London side, when all the passengers declared themselves satisfied that no ill effects had been or would be suffered.

Five of the tunnels, Kensal Green, Watford, Northchurch, Linslade, and Beechwood seem to have been constructed without undue difficulty, since no special reports on them can be traced. The remaining three, Primrose Hill, Stowe, and Kilsby, proved more difficult, particularly the classic case of Kilsby.

On the 25th March, 1835, Stephenson reported that in Primrose Hill Tunnel

"indications of more than ordinary pressure in the arch and invert in two or three places," and mentioned that he considered it unnecessary to thicken the arch, but that recent brickwork had been laid in Roman cement.

At the north end of Stowe Tunnel, the ground proved loose and subjected the timbering to heavy pressure, and the arch was given an additional 4½-in. ring. Conditions evidently got worse, as Stephenson reported on the 23rd June, 1836 that "the disturbed state of the ground at the north end of the Stowe Hill tunnel has rendered it necessary to commence open cutting. The last length taken out at that end by the ordinary methods of tunnelling clearly show from the destruction of the timbering that to persevere longer by close tunnelling would be extremely dangerous."

"Considerable risk would also be incurred in open cutting were the excavation made to the bottom of the invert at once. I have therefore, after maturely considering the matter, ordered that the excavation shall only be made to the springing of the arch, and the brickwork to that extent completed in the present manner. The arch being covered and carefully weighted by earth on the crown, the tunnelling from the interior, the undersetting of the arch, and the building of the sidewalls and invert may be proceeded with in comparative safety."

The construction of Kilsby Tunnel, the longest on the line (1 mile 666 yd) was an expensive and difficult job. It had to be constructed for more than 500 yd through a bed of quicksand, and the remaining length was also through wet ground.

That these conditions were foreseen to some extent is shown in reports from Stephenson to the Birmingham Committee. On the 21st July, 1835, he wrote "I was induced to adopt two large shafts (the 60-ft-dia. ventilating shafts), not merely on account of the great length of this tunnel, but also from a conviction that it would be found wet, which would I conceive render every possible expedient for assisting a through ventilation highly desirable." On the 22nd October, 1835, after the discovery of quicksand, and a suggestion had been made to take powers to deviate the line, he wrote "the best line through the Kilsby ridge is undoubtedly that which I originally examined by Crick, but the Union Canal, having abandoned it on account of the quicksand, I considered it prudent to take that which we have adopted. In the borings already made on that line, symptoms of quicksand made their appearance, but I believe only to a partial extent. Under the circumstances, I believe this to be the only feasible deviation that we can adopt, and shall therefore, in conformity with your instructions received this morning, request Mr Forster (Resident Engineer) to commence a thorough examination by Borings." He also suggested as a precaution, to add a clause in the Rugby contract "that this contract be considered void if the Company are compelled to adopt a deviation line."

A further letter followed on the 22nd December, 1835 which stated "Mr Forster informs me that he has forwarded to you a section showing the results of the borings on the Deviation line over the Kilsby ridge. You will at once perceive that it is fully as objectionable as the present line in reference to quicksand. I would suggest that no further expense should be incurred in securing an Act of Parliament for the proposed Deviation."

By February 1836 five shafts had been sunk; three were standing owing to water, one of these in the quicksand area, two were sunk to tunnel level (one had 42 ft of tunnel complete, the other was excavated for 33 ft) and drifts were in hand at both ends to drain away the water. The two large ventilating shafts had been commenced.

The head of the contracting firm died, and the remaining partners were anxious to terminate the contract. Stephenson agreed to this, as the job was not being pushed.

he contract was relinquished on the 12th March, 1836, and the work carried on y administration.

Stephenson set out the methods he proposed to adopt as follows:—

(a) Lay a temporary line of railway along the surface.
(b) Sink seven more shafts in the sound ground, one south of, and six north of the quicksand. These, with the three existing working shafts will divide that part of the tunnel which is free from quicksand into lengths from 200 to 220 yd long, to be worked from each shaft. Three shafts to be worked by horse gins, the others by steam engine.
(c) Continue with the draining of the quicksand and determine the number of shafts to be sunk when the draining is nearly completed.
(d) Erect a steam clay mill with kilns sufficient to supply 30,000 bricks per day, say, a total quantity of 20 million bricks.

The Resident Engineer reported to the Birmingham Committee weekly the sinkage f the water in the quicksand. The water lowered about 12 in. per week during the pring and summer of 1836, and in August two pumping shafts were sunk into the and as the water level fell. On the 8th September, No. 1 pumping shaft was eported as sunk to the bottom of the sand, showing a total depth of sand of 29 ft in.

On the 6th December, 1836, tunnelling commenced in one of the quicksand shafts, nd on the 6th January, 1837, Stephenson reported that "the first length under the uicksand in No. 1A Working Shaft was keyed in this morning at five o'clock, as this s a very important length I write on purpose to let you know it is completed." This vas followed 20 days later with a statement that "the drainage of the quicksand is ow completed as far as is necessary for tunnelling."

The opening of the railway from Tring to Denbigh Hall and from Birmingham to Rugby had to be postponed until the 9th April, 1838, and Stephenson in his report to he two Committees mentioned as examples of difficulties met with "six weeks of evere frost with only two days interval, completely shut down building of stations nd laying of permanent way. At Kilsby tunnel under ordinary circumstances eight hafts would have been amply sufficient for working the tunnel, whereas in the present ase it had been found indispensable to sink no less then twenty-five." The tunnel oday has the two large ventilating shafts and ten smaller (9-ft) shafts.

There does not appear to be a report made when the tunnel was completed, but the ine was opened from London to Birmingham on the 17th September, 1838.

Extension to Euston

The line from Camden Town, the original terminus, to Euston Grove was quite a different type of railway from the remainder of the line. The original line was laid out with a maximum gradient of 1 in 330, and heavy works were undertaken and much money spent to secure a trunk line which would permit of fast running.

The extension to Euston included of necessity a steep gradient. The terminal station was to be at ground level, and the crossing of the Regent's Canal at Camden Town controlled the level to be attained before joining the original line, and this resulted in a ruling gradient of 1 in 70. Furthermore, practically the whole length was in cutting due to the rapid rise in the ground, so that retaining walls were required for the greater part of the distance, and a number of bridges were necessary to carry roads and streets over the line (Fig. 2).

The terminus was described at the time as the "Grand Excavation," and the disposal of surplus spoil must have been a difficult problem.

In 1834 the Great Western Railway, then in course of promotion, decided to effect a junction with the London and Birmingham in the vicinity of what is now Willesden Junction, and to exercise running powers into Euston. Negotiations between the two Boards followed, and at the London and Birmingham Board Meeting on the 6th November, 1835, it was resolved "that the following terms for the admission of the traffic of the Great Western Railway be agreed to, and recommended to a special Court of the Proprietors . . . viz., for 1,750 passengers or tons of goods, or under, £15,000. For 1,000 additional passengers or tons of goods at the rate of fivepence each. For 1,000 additional, fourpence each, and all above that number threepence each. That the arrangement be terminable upon two years notice by either party." The Chairman read the foregoing terms to a deputation of Great Western Railway Directors, to which the deputation gave their assent on the part of the Great Western Railway Company, and stated that on the faith of this arrangement they would not give the Parliamentary Notices for an extension of their line into London.

Fortunately, or unfortunately, this scheme was not carried out, though land for the proposed Great Western user was purchased, and the extension built with four roads. The intention was that the western pair be used by the Great Western trains, and the eastern by the London and Birmingham.

Since the gradient was considered too steep for operating with locomotives, two 60 h.p. winding engines were erected close to the top of the incline, and the trains were hauled up by an endless rope. The descent was made by gravity, the speed being regulated by special brakesmen. Rope haulage continued until July 1844, but it was not ready for the opening of the line, as a "powerful engine" was hired for 3 months from Robert Stephenson and Company, Newcastle, to bank the trains up the incline.

Apart from the "grand but simple portico" already mentioned, the terminal buildings were not remarkable. Two platforms were provided, each about 420 ft long, roofed over for part of their length, and two intermediate sidings were laid between the main lines. All lines were connected by turntables at the buffer stops, at the end of the platform roofing and at the platform ends where a short siding at right angles to the running lines connected with another road leading to the carriage shed. In the latter were fifteen turntables serving as many transverse roads, and six more turntables were provided for adjacent lines. As all the rolling stock, including the locomotives at first, was four-wheeled, the turntables were small, but shunting must have been a lengthy process, since owing to the absence of point and crossing work, no direct movement between roads was possible.

The retaining walls were built with a curved batter, and the back of the wall was also curved; the face was broken by pilasters projecting one half-brick. Originally there must have been nearly 2 miles of these walls of varying height, but only one short length on the eastern or up-side remains today of the higher walls. The rest have been demolished during the carrying-out of subsequent widenings and alterations.

Unfortunately, these walls proved inadequate and caused considerable trouble and expense.

Professor Hosking, in a Paper[1] read before the Institution in 1844 criticized

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[1] The references are given on p. 21.

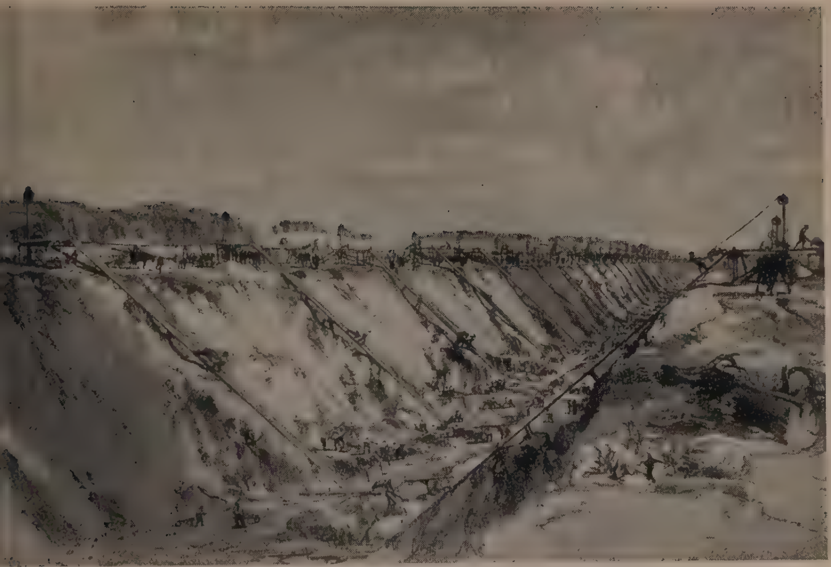


Fig. 1.—Manual excavation on the London–Birmingham Line

Fig. 2.—The "Grand Excavation"; the terminus at Camden Town

Fig. 3.—Britannia Bridge. Construction of water-span tubes

Fig. 4.—Britannia Bridge completed

counterforts behind walls and advocated placing them in front. He mentioned that the retaining walls in the cutting upon the extension of the London and Birmingham Railway from Camden Town to Euston Square were designed according to the common practice and said "it is well known that these extensive walls . . . have failed to a considerable extent. A system of strutting with cast-iron beams, across from the opposite walls, to make each aid the other, has been applied to meet the exigency."

In 1845 a Paper[2] by Thomas Hughes, advocating perforated pipes for draining cutting slopes and behind retaining walls, said "Retaining walls are frequently found to suffer severely from want of drainage, and, perhaps, no one more so than that of the Euston incline, London and Birmingham Railway." He claimed that the insertion of the pipes he advocated had proved successful in stopping movement.

Stephenson took part in the discussion, and said that the retaining walls of the Euston incline were instructive examples of the discrepancy between theory and practice under peculiar circumstances. They were designed several years since, before he had attained his present experience of the effects of the London Clay.

How many engineers could say the same of London Clay since that date?

Britannia Bridge

Whilst the London and Birmingham Railway is an adequate memorial to Robert Stephenson's ability, it did not provide an opportunity to demonstrate his skill in the design of large bridges. This chance came later when he was appointed Engineer of the Chester and Holyhead Railway, which necessitated the construction of two bridges of large span, that at Conway of 400 ft, the other over the Menai Straits with two spans of 450 ft. Both bridges are over tidal waters, and the sites precluded erection on falsework owing to navigation and rapid tidal currents.

The first application for Parliamentary powers was made in the Session of 1843–44. The chief engineering work then involved was the bridge over the Conway, close to Telford's existing suspension bridge. The crossing of the Menai Straits was intended to be achieved by taking over one of the roadways of the suspension bridge for railway purposes. As this bridge was considered inadequate to carry trains with locomotives, it was intended to convey the trains across in a divided state if necessary, by means of horse traction, another locomotive being in readiness to be attached on the other side. Thus the passage of engines was entirely obviated. The Commissioners of Woods and Forests assented to the proposal, but with the condition that the use of the south roadway for railway purposes should only be temporary. The Company therefore had to abandon this part of their plan and to propose an independent bridge for the railway. The Bill was permitted to pass, and the Company instructed their Engineer to deviate the line and select the best site for crossing the straits by an independent bridge.

Stephenson selected the site known by the name of the Britannia Rock, about 1 mile south of Telford's suspension bridge. He proposed to construct a bridge with two cast-iron arches, each of 350-ft span, the roadway being 105 ft above high water ordinary spring tides.

At Britannia Rock, instead of two arches being erected on two abutments and one pier, the scheme was to erect three balanced cantilevers on three piers. The arches were to be built by placing equal and corresponding voussoirs on opposite sides of the pier at the same time, tying them together by horizontal tie-bolts.

In November 1844 the Company deposited new plans preparatory to asking for powers to carry out the deviation.

The proposed bridge was opposed by the shipping interests, as rendering navigation more dangerous by restricting vessels to a narrower channel, and stating that the massive piers and spandrels would shelter vessels from the wind in situations when it was of the utmost importance to them.

These objections seemed likely to endanger the passage of the Bill, so Stephenson reconsidered the possibility of stiffening the deck of a suspension bridge so as to make it suitable for railway traffic at high speeds.

This led to the idea of a huge wrought-iron rectangular tube, so large that trains could pass through it, with suspension chains on each side, and this, in turn, to the tube as a beam with the chains as auxiliaries.

Parliamentary opposition led to the Admiralty being asked for a report on the proposed bridge over the Straits. The report confirmed the site as the most suitable one available, but stated the proposed arch design would be detrimental to the interests of navigation. It said that the pier on the Britannia Rock should not be more than 50 ft square, and be built on the highest part of the rock. Further, no pier or abutment should be erected on either side of the Straits projecting beyond the line of ordinary high water mark, and there should be a clear headway of 105 ft above high water ordinary spring tides at every part under the proposed bridge where vessels now passed.

Stephenson decided that a tubular bridge was the only structure which combined the necessary strength and stability for a railway. He informed the Directors of the Chester and Holyhead Railway Company he was prepared to carry out a bridge of this description, and gave evidence to this effect before a Committee of the House of Commons on the 5th May, 1845. He states "the evidence which I gave before the Committee . . . was received with much evident incredulity so much so that . . . the Committee stated that they would require evidence, and especially that of the Inspector General of Railways before they could pass the Bill authorizing the erection of such a bridge."

The Preamble of the Bill was passed, but a resolution come to, which left the question of the bridge entirely open for further consideration. The Bill passed the Parliamentary Committee and received the Royal Assent on the 30th June, 1845.

An important series of tests on the strength of various tubular constructions was now entered upon. The performance of the experiments was left to Mr Fairbairn, and the beams were fabricated and tested in his shipyard at Millwall. Mr Hodgkinson, the eminent scientist and mathematician, also collaborated on the tests.

Twelve experiments were made on cylindrical tubes, seven on elliptical, followed by fourteen on rectangular tubes. These tests were spread over the period from the 6th July to the 14th October, 1845. The tubes were of different lateral dimensions and spans, and were constructed of plates of different thicknesses. All were tested to failure by a concentrated load at mid-span.

Stephenson reported to the Directors on the tests on the 9th February, 1846 "the object of this investigation . . . was to test the truth of the views I entertained respecting the employment of a large wrought iron tube instead of cast iron arches as was originally proposed, but which we were compelled to abandon in consequence of the Admiralty refusing to allow the erection of such a structure from the belief that it would injuriously interfere with the navigation of the Straits."

"In the course of the experiments, it is true some unexpected and anomalous results presented themselves, but none of them tended in my mind to show that the tubular form was not the very best for obtaining a rigid roadway for a railroad of a

span of over 450 ft, which is the absolute requirement for a bridge over the Menai Straits."

"Another instructive lesson which the experiments disclose is that the rectangular tube is by far the strongest and that the circular and elliptical should be discarded altogether."

Stephenson considered some further experiments were required, but "in the meantime, however, as I consider the main question settled, I am proceeding with the designs and working plans for the whole of the masonry, which I expect to have the pleasure of submitting to you in a fortnight from this time."

Accompanying this report were separate reports from Messrs Fairbairn and Hodgkinson giving details of the tests.

The last series of experiments, from July 1846 to April 1847, were made on a large model, one-sixth full size. Six tests were made, and after each one weak points in the model were made good. At the finish, Mr Edwin Clark, the Resident Engineer wrote "the magnificent model . . . failed at length from the crushing of the top, after carrying a greater weight than even a double line of locomotives throughout the whole length. Nothing could be more satisfactory than this result; an addition of material of only 1 ton to a beam weighing originally only 5¼ tons, having increased the breaking weight from 35½ tons to upwards of 86 tons."

The final design consisted of two wrought-iron tubes each carrying one track. There are two land spans of 230 ft and two over water of 459 ft each, the overall length of each tube being 1,511 ft. The depth of the tubes increases from 22 ft 9 in. at the ends to 30 ft at the central Britannia tower, and each has a width of 14 ft. 9 in.

Both upper and lower chords are of cellular construction. The upper is of eight cells 21 in. × 21 in., and the lower of six cells 2 ft 4 in. × 21 in. This form of construction was adopted for the top member to obviate any risk of buckling, particularly when the tube was being jacked up into position as a simple span. The bottom member is in compression at the three towers, as the tubes are continuous over the the four spans, and a cellular form is also desired to reduce the length of the rivet grip.

The tubes are supported on two abutments and three towers of masonry, and are carried through the towers partly on roller bearings and partly suspended from spade bolts hung from movable cast-iron beams, with the exception of the Britannia tower, which has fixed bearings. The towers are carried above the tubes so as to carry suspension chains originally intended for support during erection or possibly permanently.

As the estimated weight of each water span of the tubes was 1,285 tons and its length 472 ft, the method of erection had to be decided upon before the site for its assembly could be selected.

The scheme originally favoured by Stephenson was to construct a suspension bridge of sufficient strength to carry the tube, the roadway of this bridge being at the intended level of the tubes, and to construct platforms at the same level at each approach to the suspension bridge, and on these platforms and on the bridge to lay a railway. Then to construct the tube on the railway on a line of trucks, load the suspension bridge with a train about equal in weight to that of the tube and draw the tube on to the bridge at one end simultaneously with the withdrawal of the line of loaded trucks at the other so as to prevent any great undulation in the suspension bridge. The chains of the temporary bridge would be available as additional support for the tubes if desired.

Plans for the masonry were ready on the 17th March, 1846, and the first stone in

the foundations was laid on the 10th April by the Resident Engineer for the masonry, Frank Forster. Work went on steadily; the side towers were completed on the 22nd February, 1849, and Stephenson laid the last stone on top of the Britannia tower on the 22nd June in that year.

A quotation for the supply of the suspension bridge was received, the figure being much higher than expected, and as it was intended to build the tubes to be self-supporting in any case, discussion of schemes for erection was resumed in the hope of dispensing with the suspension bridge.

Mr E. Clark, the Resident Engineer, noticed contractors raising a small water tank at Crewe by jacking, and thought the same method might be used for raising the bridge tubes. Stephenson finally agreed to using this method, and the towers were built with vertical recesses to receive the ends of the tubes when floated into position.

The tubes for the 230-ft land spans were built in situ on temporary timber staging. The staging on which the four water-span tubes were built was constructed on the Caernarvon shore (Fig. 3, facing p. 17). When assembled, these tubes were floated on eight pontoons and warped into position, being landed in the recesses in the towers at or about high water.

The control of operations when the tubes were afloat was given by Stephenson to Captain Claxton, R.N., who recruited a number of sailors in Liverpool. The scheme was to warp the pontoons bearing the tube out from the shore, swing the tube by the tide, and let it drift towards the towers, getting it into proper position by lines controlled by capstans mounted on the shore, and depositing it on the masonry by the falling tide. The rapid currents in the Straits made this an exciting and hazardous operation, but all the tubes were successfully landed.

They were raised to their correct level by hydraulic rams fixed near the top of the towers, the tube being suspended on a multiple link chain at each end. The rising tube was followed up by timber packing in the centre and by brickwork in Roman cement at each side. The ram stroke was 6 ft, and lifted the tube at the rate of approximately 2 in./minute.

The importance of following the rising tube with packing was demonstrated on the 17th August, 1849, when the press at the Anglesey end of the first tube to be launched failed by the bottom of the cylinder breaking away from the body, the tube dropped on the timber packing which was severely crushed, and the special lifting frames and beams at the tube end were severely damaged. The bottom plates of the tube were bulged inwards 3 to 4 in, and the curvature extended for 40 ft along the tube. All was made good, and lifting was resumed on the 1st October, and the final elevation was reached on the 13th. The junction piece carrying the tube across the tower was riveted to it and the tube lowered on its permanent bed in the Anglesey tower on the 9th November.

The second tube between the Caernarvon and Britannia towers was floated on the 4th December and set on its permanent bed on the 7th February, 1850. The two tubes above mentioned now form part of the up line.

To develop the support moments so that the bridge would act as a continuous girder, the tubes were tilted before riveting to the junction pieces and afterwards lowered on their permanent bearings, thus causing a negative moment at the towers. It seems, however, that Stephenson did not fully attain his object, as the moments at mid-span are greater than those over the support.

This operation was completed on the 4th March, 1850, and on the following day three locomotives passed through. Stephenson and the contractor drove the last rivet and a train of three engines, forty-five loaded wagons, and carriages containing

700 persons, weighing in all 503 tons, passed through the tubes to Holyhead. The Government inspection took place on the 15th March, and the line opened for public traffic on the 18th. The superstructure for the down line was then proceeded with, and opened in October 1850 (Fig. 4, facing p. 17).

The abutments were similar in plan, but owing to the configuration of the ground the Caernarvon one is 88 ft high, whilst the Anglesey one is 143 ft. The lintels over the entrances to the tubes are single stones 20 ft long, both on abutments and towers. At the ends of the abutment wing-walls are colossal lions couchant on pedestals. Their length is 25 ft and each weighs 30 tons. Each is composed of eleven pieces of limestone, and they are 12 ft in height. They were designed and executed by Mr Thomas, who was engaged on sculpture for the Houses of Parliament. He also designed a colossal figure of Britannia for the centre tower of the bridge, but its great cost prevented its construction.

The total masonry in the bridge is 55,265 cu. yd, weighing 104,875 tons. Almost half of this mass of material was brought from quarries in Anglesey, opened and worked by the contractors for the masonry. The average height to which it has been raised is 80 ft, and $2\frac{3}{4}$ years were occupied in its erection. Throughout that time it was set at the rate of 3 cu. ft/min; from 500 to 600 men were continuously employed on the erection, and a further 300 to 400 in the quarries and in bringing the stone to the Straits. The weight of wrought ironwork in the bridge is 9,360 tons and that of cast iron 1,987 tons. The cost of the bridge was, masonry £158,704, ironwork and erection £443,161—a total of £601,865.

The bridge has given continuous service without alteration to the present date. Careful maintenance has been continuous, and apart from the renewal of a few of the spade bolts, repairs have been negligible.

Another tubular bridge was designed and constructed by Stephenson for the Grand Trunk Railway over the St Lawrence river at Montreal in 1859. The type is now obsolete, due largely to advances in the iron and steel industry, both in the metallurgical and manufacturing fields.

I hope the foregoing shows how fortunate this country was in possessing an engineer such as Robert Stephenson at the begining of the railway era.

In conclusion, I wish to thank Mr J. Taylor Thompson, M.C., M.I.C.E., Chief Civil Engineer, London Midland Region, British Railways, and Mr F. B. Belton, B.Sc. (Eng.), A.M.I.C.E., of his staff for access to plans and other information, and to Mr L. C. Johnson, the Archivist of the British Transport Commission and his assistant Mr J. M. Campbell for access to original documents and records.

References

1. W. Hosking, "On the introduction of Constructions to retain the sides of deep Cuttings in Clays, or other uncertain soils." Min. Proc. Instn Civ. Engrs, vol. 3, p. 355 (1844).
2. Thomas Hughes, "Description of the Method employed for Draining some Banks of Cuttings on the London and Croydon, and London and Birmingham Railways; and a part of the Retaining Wall of the Euston Incline, London and Birmingham Railway." Min. Proc. Instn Civ. Engrs, vol. 4, p. 78 (1845).

Mr A. C. Hartley, Vice-President, proposed the following resolution:

"That the best thanks of the Institution be accorded to the President for his Address and that he be asked to permit it to be printed in the Proceedings of the Institution."

Saying that it was a great pleasure to him that his first duty as a Vice-President should be to express the thanks of the Institution to the President for his Address. The immediate Past-President has told of the great services to the Institution of the new President on committees and in national and international work. Mr Hartley had had the pleasure of serving with him on some of those committees, and had always enjoyed the way in which his remarks had enlivened and encouraged the proceedings at times. Mr Hartley had also sat under his chairmanship on some of those committees, particularly on the Building Research Board, and had always admired very greatly the extremely efficient way in which he had conducted the affairs of the meeting and got the right answers in the minimum of time.

The President had clearly taken great pains with the preparation of his Address, and had delivered it in a fascinating manner. Members were indebted to him for bringing out so clearly the great efforts of the early members of the Institution, who carried out such great works, some of which would be tremendous undertakings today, even with modern plant.

Mr M. G. R. Smith, seconding the resolution, said that it was a great ordeal for the President, within a few moments of his taking the Chair, to be asked to give a Presidential Address, and members were indebted to Mr Wallace for the interesting manner in which he had done so.

Mr Watson, the retiring President, had said that he would not refer to Mr Wallace's professional career, but Mr Smith ventured to mention two activities which were of an outstanding pioneering nature. One was that he understood that Mr Wallace had been the instigator of the development of the internal combustion engine in relation to light-weight vehicles, and therefore it might be said that he was one of the pioneers of the present policy of British Railways in adopting the light-weight Diesel traction. He had also pioneered the introduction of flat-bottom rails on British main lines.

The President acknowledged the resolution and gave permission for the Address to be printed in the Proceedings.

SUPPLEMENTARY MEETING

13 October, 1955

Mr H. J. F. Gourley, Vice-President, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Institution were accorded to the Authors.

Paper No. 6098

THE RAISING AND STRENGTHENING OF THE STEENBRAS DAM †

by

*** Solomon Simon Morris, B.Sc.(Eng.), M.I.C.E., M. Amer. Soc. C.E.,**

and

William Scott Garrett, B.Sc., A.M.I.C.E.

SYNOPSIS

At the time when a serious water shortage confronted Cape Town investigation revealed that the Steenbras Dam, the City's main source of water supply, required strengthening because of inadequate spillway capacity. Construction of a new £6,000,000 dam and pipeline had already been initiated but immediate interim measures were essential to avoid the imposition of drastic water restrictions which would retard the City's development.

The raising, as well as strengthening, of the Steenbras Dam offered an effective solution to the problem, and analysis of a number of alternative methods showed that the Coyne process of anchoring the dam by post-stressing would have every advantage, both in cost and expedition.

Essentially the process is one of placing vertical cables through the wall of a mass-concrete dam from the crest into the foundation and stressing the cables to produce stabilizing compressive forces on the upstream face.

Principal items of construction were drilling the ducts for the cables, homing-in the cables, and anchoring and tensioning them. Although the actual volume of work involved was not so heavy as that necessitated by orthodox methods considerable demands were imposed on the specialized knowledge and skill of the contractor. Instead, therefore, of following the normal procedure of calling for tenders a contract was negotiated.

The most interesting design problems were the determination of the required depth and length of the cable anchorages, for which semi-empirical methods were used.

Extracts from the specification covering the vital operations of drilling and placing, and the tensioning of the cables are quoted; and a brief description is given of the manner in which the work was actually executed.

Construction began on the 1st March, 1953, and was completed in October 1954, the work schedule having been planned to fit in with the seasonal rise and fall of the reservoir level. A critical factor in the planning of the work was the difficulty of access throughout the narrow 1,400-ft-long crest of the dam.

Because of difficulties created by obstructions in the dam wall and the alternating character and dip of the rock foundation, diamond replaced percussion machines for most of the drilling.

* Mr Morris is City Engineer, Cape Town, and Mr Garrett is Manager, The Cementation Company (Africa) (Pty), Ltd.

† This Paper was also read at a meeting of the South African Institution of Civil Engineers on the 25th October, 1955, and will be published in the Transactions of that Institution, vol. 5, No. 10, October 1955.

Homing-in of the cables presented little difficulty but grouting for anchorage posed a number of problems. Extensive full-scale experiments were carried out before the method finally adopted was developed.

In spite of inevitable construction hazards work proceeded satisfactorily close to programme and was completed in time to store additional winter water. The project has proved to be an unqualified success from every angle. Indeed, sale of the additional water which the raising of the Steenbras Dam made available to Cape Town in the first summer after completion of the work more than sufficed to cover the complete cost of construction.

Hydrological and planning aspects

The water problem in Cape Town

Water undertakings, no less than other public works, suffer universally from the shackles of red tape and inertia. Time and again it has needed almost a major catastrophe to ensure the initiation of new projects or the augmentation of existing works. This state of affairs is almost inevitable where the execution of works depends on a multiplicity of authorities. The Cape Town Waterworks undertaking is no exception to the general rule; South African law requires that in addition to a local authority's approval for new capital works the acquiescence of the Provincial and Central Government authorities must also be obtained.

The storage, supply, and distribution of water in Cape Town are the responsibility of the City Engineer's Department. The Department not only supplies the needs of the half million citizens who reside in the 80-sq.-mile municipal area, but delivers water in bulk to fourteen adjoining local authorities spread over an area of about 300 sq. miles.

Since the sole object is to deliver pure and potable water as cheaply as possible and since service and not profit is its primary objective the activities performed by the undertaking are virtually those of a Water Board.

In recent years South Africa has enjoyed an unprecedented expansion in trade and industry. Cape Town, mother city of the Union, has played no mean role in this national development and has had its full share of the problems normally associated with the rapid increases of urban population which invariably follow industrial growth.

One of the most serious of these problems arose from the substantial increases in water used during the war and post-war periods.

In 1939 the City consumed, on an average, 13·1 m.g.d. (million gallons a day). During the war consumption rose sharply, reaching 19 m.g.d. in 1945; and in 1950, when one of the Authors was appointed City Engineer (and as a result assumed responsibility for the City's water supply) consumption had reached 21·5 m.g.d.

Throughout this period no additions were made to the City's sources of supply, and assured yield from all sources remained static at a figure of 23 m.g.d. The most important source of supply was Steenbras, a mountain catchment about 30 miles from Cape Town, yielding an assured supply of 20 m.g.d.

As was anticipated, it was not long before demand overhauled assured supply. By 1953 consumption had risen to 25·2 m.g.d., i.e., 2·2 m.g.d. in excess of assured supply.

Water-augmentation schemes

The need for a water-augmentation scheme in Cape Town had, of course, been obvious for a long time, and in fact by 1950 the Union Parliament had before it a Bill

aimed at granting the city the necessary powers to develop a new water source in the Wemmershoek catchment—a mountainous area of high rainfall about 45 miles from Cape Town.

To expedite progress and to bring this £6,000,000 scheme to speedy fruition the City Council resolved, in 1951, to vest in a specially appointed Board of Engineers the responsibility for the technical supervision of all work connected with the augmentation of the city's water supplies. This board was duly constituted in March 1951 and comprises Mr S. S. Morris, City Engineer, Cape Town, as Chairman, with Dr J. L. Savage of Denver, Colorado, United States, and Mr N. Shand as members. The Board's functions are primarily directive and supervisory; all detailed investigation and design as well as site supervision of construction are carried out by the City Engineer's Department.

It was clear to the Board of Engineers from the outset that the Wemmershoek scheme could not be constructed in time to prevent consumption exceeding the then assured supply; obviously there would be an interim period of several years when, if dry weather conditions were encountered, water shortages might be felt in the City. The course of events was shortly to prove the validity of these expectations, and as a result of the unusually dry summer of 1953/54 the City experienced its first water restrictions for many years.

One of the first directives of the Board of Engineers, therefore, was that investigations be made of possible stop-gap measures to alleviate the position during the interim period. Of these, one of the most promising appeared to be a temporary raising of the Steenbras Dam. Associated with this was the information placed before the Board that the spillway capacity and, in consequence, the stability of the dam were not as great as might be desirable; indeed the adequacy of the spillway had been questioned as far back as 1932 in the discussion on a Paper by Lloyd-Davies.[1]

Peak flood and spillway capacity

The Steenbras spillway, of the ordinary overflow type, 222 ft long, was crossed by a footbridge whose soffit was 2 ft above the spillway crest. Discharge capacity with a surcharge of 2 ft (i.e., when the water would be touching the soffit of the footbridge) was 2,270 cusecs. Three scour pipes combined to raise the total discharge to about 2,800 cusecs, equivalent to an inflow of only 107 cusecs per sq. mile (the catchment area being 26·2 sq. miles).

Records showed that the design capacity had been exceeded on four occasions since completion of the spillway in 1927, i.e., an average of once every 6 years.

The maximum flood, which occurred in September 1944, produced a 3-ft 9-in. surcharge, corresponding to a discharge of 3,800 cusecs. Calculations (based on the hydrograph registered by an automatic recorder and corrected for reservoir lag) indicated that this corresponded to a peak inflow of about 7,300 cusecs, equivalent to 275 cusecs per sq. mile.

Flood data from other catchments in the Western Province indicated that greater floods than this had occurred elsewhere in that area, and it was not unreasonable to assume that a substantially higher flood than 275 cusecs per sq. mile might well occur at Steenbras.

Analysis of previous floods has indicated that frequency of occurrence agrees fairly well with curves of flood probability given by the Union Department of Irrigation.[2] These curves show a value for the 100-year flood of 17,000 cusecs (650 cusecs per sq.

[1] The references are given on p. 48.

mile), and this figure also appeared reasonable in relation to other flood data available to the Board of Engineers. Although a later Paper by Mackenzie [3] indicated a 100-year flood of substantially less magnitude, actual observations at Steenbras fitted the data given in the earlier Paper [2] better. A peak inflow of 17,000 cusecs was therefore adopted as a basis for design.

For calculating the discharge corresponding to a peak inflow of 17,000 cusecs, the inflow hydrograph given in the first Paper of the Irrigation Department [2] was used.

On this basis a flood of the above magnitude would have overtopped the dam for its whole length—an obviously undesirable condition calling for expeditious remedy.

As an immediate palliative the footbridge was removed. With this obstruction eliminated, the design flood would have caused a discharge of 10,000 cusecs, with a surcharge of 5 ft 6 in.

Existing structural conditions

Calculations showed that on the basis of uplift varying from two-thirds hydrostatic pressure at the upstream face to zero at the downstream face, the resultant force would intersect the middle third when the surcharge was 2 ft.

Admittedly a surcharge as much as 3 ft 9 in. had already occurred without any untoward effect; this, however, was considered to be an encroachment on the factor of safety, rather than an indication that design assumptions regarding uplift had been conservative. Keener [4] has indicated that in modern dams, with their vertical drains near the upstream face, and first-class construction-joint treatment, there may often be surprisingly little pore pressure; but it would have been unduly optimistic to have applied this reasoning to the Steenbras Dam. Although some upstream-face drains were provided, evidence of joint leakage indicates that treatment was not perhaps as effective as it could be with modern technique.

In addition to calculations of stability based on two-thirds-uplift assumption and middle-third criterion, calculations were also made using the "extreme load" criterion of the United States Bureau of Reclamation. [5] This defines the limits of a crack or open construction joint that may develop adjacent to the upstream face, and requires that the resultant force with full uplift allowance should not be such as to overstress the toe.

The above investigations indicated that the spillway section of the dam under a 5-ft 6-in. surcharge might well have been unstable, the resultant force passing far outside the middle third with the basic uplift conditions as noted above, and altogether outside the section using the "extreme load" criterion. Under a similar analysis it appeared probable that the non-overflow section of the dam was also unstable, the resultant force being well outside the middle third of the base in the one case, and only just within the section using the "extreme load" criterion. (It is of interest to note that calculations on the "extreme load" basis indicated that for a 3-ft surcharge, since the resultant was well within the section, both the spillway and non-overflow portions of the dam were probably stable—a conclusion confirmed by experience).

The possibility of extreme flood conditions occurring could not be rejected and, since the consequences of failure of the dam would have been disastrous, remedial measures had to be taken. In the planning of such remedial measures it was obviously desirable to consider the benefits that would accrue if additional storage capacity was provided by raising, as well as strengthening the dam, particularly since the existing filtration plant and trunk mains were fortunately of sufficient capacity to handle the increased flows that would result from additional storage and higher draw-off.

Benefits of increased storage—interim period

Since the Wemmershoek Scheme would probably not be in operation until the summer of 1957-58 the immediate benefits to be derived from any interim scheme were perforce limited to the three summers from 1954–55, 1955–56, and 1956–57. Moreover, such benefits would be dependent on the probability of winter rains refilling the reservoir to its new raised height. Computation from available run-off data in the catchment yielded the order of probabilities indicated in Table 1.

TABLE 1.—PROBABILITIES OF FILLING DAM TO VARIOUS HEIGHTS

Height of raising: ft	Probability of complete filling to new height
0	0·45
4	0·44
6	0·41
9	0·38
12	0·31

Calculation showed that because the variability of flow of the river was large by comparison with the increment of storage involved there was little difference in the probabilities of filling for raisings of various heights up to say 10 ft.

Estimated consumption over the interim period before Wemmershoek would be in commission indicated that unrestricted supplies could be maintained with reasonable certainty only by a raising of about 15 ft and on the assumption that the reservoir filled to the new level during the winter of 1954.

A 10-ft raising gave a 90% probability of avoiding restrictions and a 6-ft raising an 80% probability. If no raising was carried out restrictions were inevitable. Indeed in the season prior to completion they were perforce imposed. The figures showed that a raising of say 6 ft offered a significant improvement in the chances of avoiding restrictions, but any raising over about 6 ft was very much subject to the law of diminishing returns.

Benefits of increased storage—long-term considerations

If the raising were undertaken as a permanent measure, then not only interim benefits, but also a permanent increase in assured supply would accrue.

The amount of increase is shown in Table 2.

TABLE 2.—ADDITIONAL SUPPLY RESULTING FROM RAISING DAM TO VARIOUS HEIGHTS

Height of raising: ft.	Additional supply: m.g.d.
3	0·67
6	1·35
9	1·90
12	2·40
15	2·85

From a long-term point of view it might appear that a raising of as much as 15 ft would have been justifiable, but once again detailed investigation showed that increasing costs made raising above 6 ft of little net benefit.

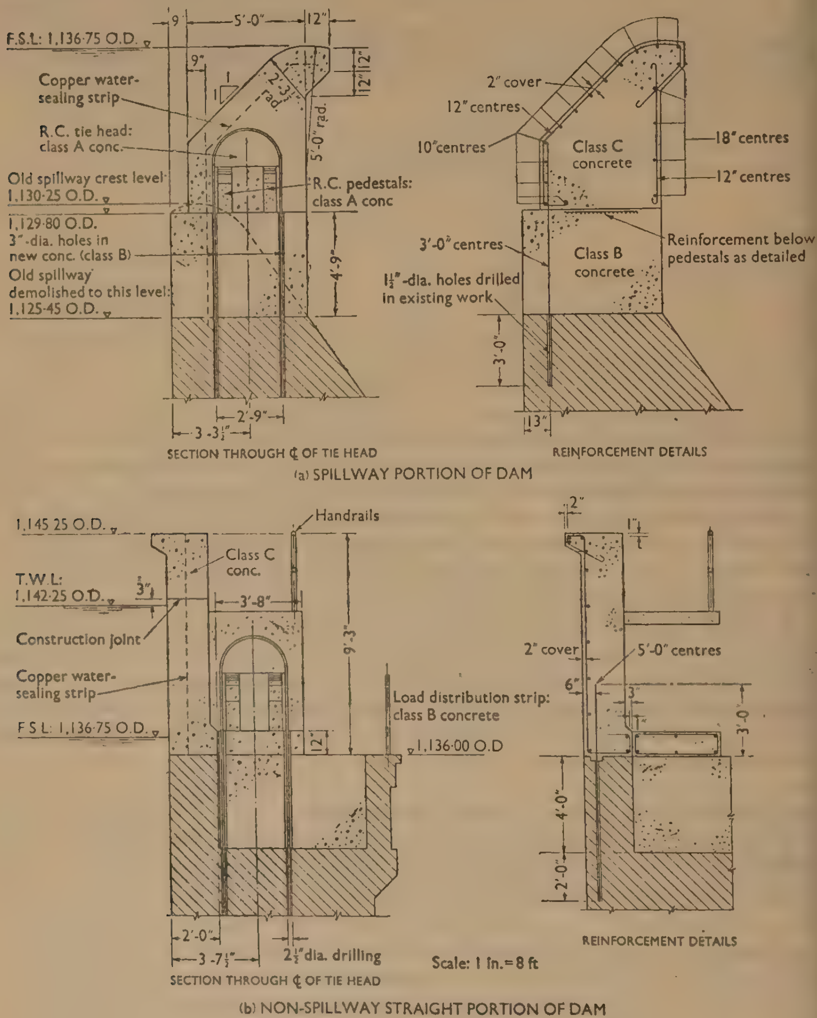
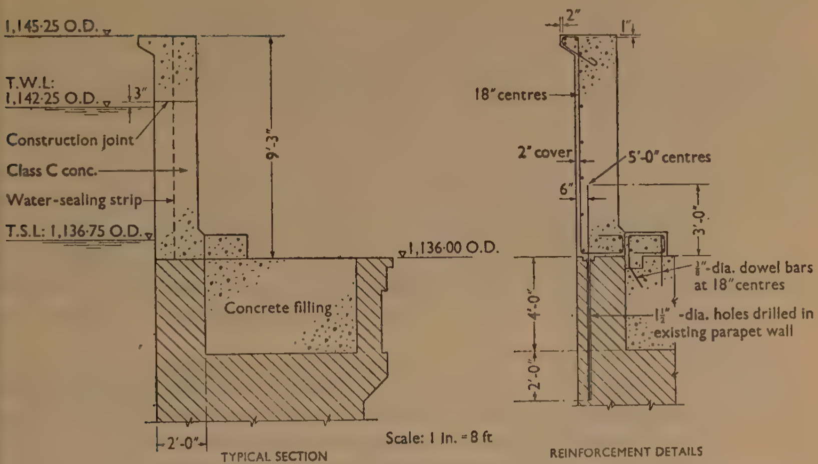
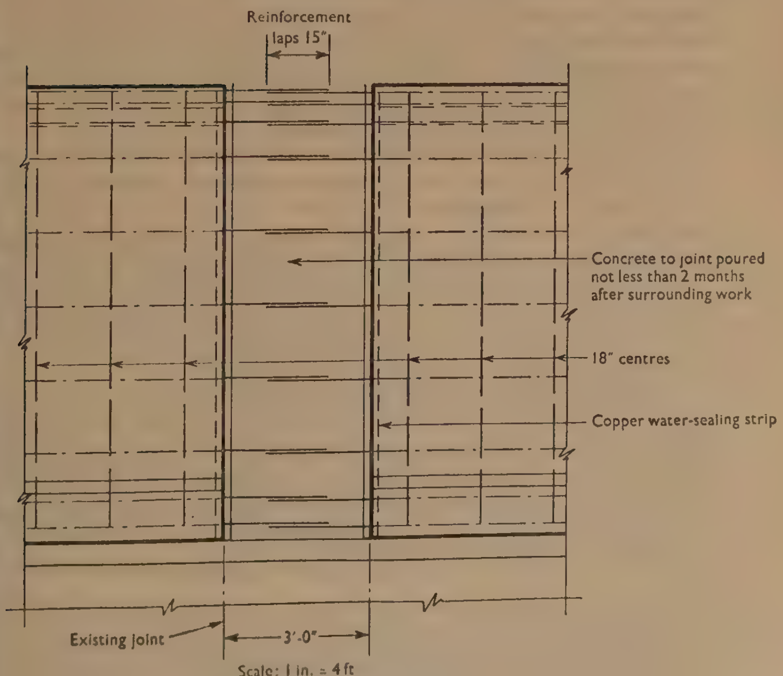


FIG. 2.—REINFORCED CONCRETE DETAILS

Another long-term factor considered was that at some time in the future, when the Wemmershoek scheme has become fully exploited, one of the possibilities of obtaining further supplies will be the diversion of streams from adjacent catchments into the Steenbras reservoir. This would call for a major raising of the Steenbras wall, and in designing remedial measures this possibility could not be ignored.



(c) NON-SPILLWAY CURVED PORTION OF DAM



(d) CONSTRUCTION JOINTS IN RETAINING WALL ON CURVED PORTION OF DAM ONLY

FIG. 2.—REINFORCED CONCRETE DETAILS

Alternative remedial measures

Important and valuable though additional storage might well prove, the Board's primary consideration in assessing the merits of alternative measures was the need to ensure stability of the dam against maximum possible flood.

The first investigations made, therefore, were on purely remedial measures, the following studies in particular being carried out:—

(1) *Cutting down the existing spillway by* 3 *ft* 6 *in*.—Although this was certainly the easiest, quickest, and cheapest method in itself, it would have involved a loss of about 810 million gal in storage and about 0·80 m.g.d. in assured supply. This constituted a very serious disadvantage, particularly in view of the difficulties anticipated for the interim period prior to construction of the Wemmershoek scheme. Cutting the spillway was therefore not recommended.

(2) *Gravity strengthening*.—This would have involved thickening the dam on its downstream side so as to permit the water level to rise to 5 ft 6 in. above the spillway crest with safety. The work would have been difficult to carry out and, under the rather restricted site conditions, would have been slow in execution. The cost would have been of the order of £75,000 to £80,000.

(3) *Strengthening the dam by prestressing cables* (*The Coyne method*).—This method—the one finally adopted and fully described in this Paper—was admittedly unorthodox and had hitherto been confined to works under the direction of Mr André Coyne in France and in the French Empire; although it was noted that the method had been proposed for the strengthening of the Tansa Dam near Bombay. It offered, however, sufficient advantages to render it worth considering very seriously.

Construction would be rapid and there was every indication that savings would result over other more orthodox methods. There would be no interference with amenities, in particular with the rock gardens immediately below the dam, which have become an outstanding attraction and have proved a constant source of pleasure both to the citizens of Cape Town and to visitors. It would have the further advantage of being a permanent asset in the event of future major raising, since the prestressing forces can be taken into account in stability calculations, no matter what method is adopted for further raising.

A careful preliminary survey of the process was therefore made by the Board of Engineers. Inspection at the site led to the conclusion that the foundations would be satisfactory, and had sufficient strength to provide the necessary anchorage for the cables. In this connexion the Board of Engineers was particularly fortunate in that one of its members had been associated with the construction work in a previous raising of the Steenbras Dam, and was able to give positive assurance of the quality of foundation rock.

Only one point emerged on which there was doubt, namely, the possibility of corrosion of the prestressing cables which might be produced by the soft acid brown water of the Steenbras catchment. This question was reserved for further investigation. After inspection of conditions at Steenbras, proposals were put forward which gave satisfactory assurance that this danger could be avoided.

The Board of Engineers obtained preliminary estimates from the firm

later selected as contractor, which indicated that the cost of purely remedial measures—that is, merely the strengthening of the dam without any raising—would be approximately £46,000.

(4) *Gate-controlled additional spillway.*—A new gate-controlled spillway, providing additional capacity, could have been constructed in a channel excavated adjacent to the left flank of the dam. This proposal would have been simple and reasonably expeditious in its execution; and construction would have presented no difficulty. The cost was estimated to be of the order of £70,000, and a large part of the rock gardens below the dam would have been destroyed. Furthermore, the works would not have been a fully permanent asset in the event of a future major raising of the dam, since a large part of the gate excavation would then have had to be filled with concrete, and only the gates themselves would be recoverable assets.

(5) *Siphon spillway.*—The construction of a reinforced concrete siphon spillway over the existing spillway crest would have been a relatively straightforward operation. The cost was estimated at about £30,000, and construction itself was expected to be reasonably expeditious. There were, however, a number of disadvantages.

Model tests showed that, because of the unusually large wave action which occurred at Steenbras, difficulties would probably be encountered in operating the system, especially in priming and depriming. These difficulties could have been overcome, but the necessary investigations would have occupied a good deal of time, and might have delayed commencement of work in circumstances when the time factor was of paramount importance. In the event of a major raising of the dam the siphon would not be an asset, since it would either have to be demolished or filled with concrete.

lternative proposals: augmentation of reservoir capacity

The choice of alternative proposals for remedial measures, therefore, lay between ne Coyne method and the siphon spillway. Had it merely been a question of creasing the capacity of the spillway and improving the stability of the dam, the dvantage may well have lain with the siphon. On the other hand the additional orage capacity easily and economically made available by the Coyne process afforded ne possibility of very substantial benefits, both for the interim period prior to the ompletion of the Wemmershoek scheme, and as a long-term improvement.

To enable the Board of Engineers to determine the optimum raising, preliminary timates of cost were prepared for various raisings from 3 ft to 30 ft, with a more etailed examination for a 6½-ft raising, the cost of which was £95,000. With these gures in their possession, combined with the hydrological data described above, the oard concluded that a 6½-ft raising by the Coyne system would offer the greatest dvantages.

The scheme adopted envisaged a free spillway of the existing length, with a rise water level of 5½ ft over the crest during peak flood conditions, provision being ade at the same time for adequately strengthening the structure to withstand these nditions. It would thus be feasible, as a second stage of augmentation, to provide siphon spillway, instead of one of the ordinary overflow type, whereby the 5½ ft of ood-capacity height could be converted into 4 ft additional storage, with an allownce of only 1½ ft for flood rise. Alternatively, additional temporary capacity,

aimed merely at meeting the requirements of the interim period, might be provided by such expedients as temporary brickwork, sandbagging, or flashboards.

Strengthening of the wall by the Coyne system, combined with an immediate raising of $6\frac{1}{2}$ ft, thus offered considerable advantages provided that the work was carried out properly and expeditiously.

If full advantage was to be taken of the raising before the Wemmershoek scheme became effective, extra storage was required at Steenbras not later than the winter of 1954. If this could be achieved there was thus the attractive possibility that the Steenbras wall might be strengthened, the City of Cape Town saved from temporary water shortage, and at the same time the whole capital cost of the strengthening and raising might be recouped by the sale of the extra water made available.

Award of contract

The work to be carried out, however, was far from orthodox; it involved not only the highly specialized technique needed to anchor and stress the cables, but also the specialized ability to drill holes to receive the cables from the crest of the wall into the rock wall below the dam foundations. These holes had to be drilled to within close tolerances for straightness and direction, and they had then to be rendered watertight. Finally, after the cables had been placed, it was vital that they should be sealed completely to prevent corrosion.

In view of the importance of the time factor and the high degree of specialization required to perform the work, it was clear to the Board of Engineers that the normal procedure of advertising for competitive tenders could not be followed. If the raising was to be carried out in time to exploit the benefits hoped for, it was essential to negotiate a contract immediately with a contractor experienced and skilled in the highly specialized operations involved.

Fortunately a firm had been established in South Africa which met these requirements. This Company moreover had, together with its parent company in Britain, already worked in close co-operation with Mr Coyne. During the early stages of the investigations it had prepared estimates for doing the work by the Coyne system and submitted a fairly detailed scheme for the $6\frac{1}{2}$-ft raising.

Since, so far as the Board was aware, there were no other firms in the country equally equipped to perform the tasks involved, the Board recommended to the Council that a contract be negotiated with this firm. This recommendation was accepted by the Council on the 24th January, 1953.

Having agreed that the contract for the main raising and strengthening was to be a negotiated one, it was decided that all ancillary works, and indeed all works which were not part of or closely associated with the stressing, should be excluded from it. These ancillary works comprised raising the valve tower and access bridge thereto, extending the main wall to the valley sides on either flank of the dam, and certain minor road works. Separate contracts under normal competitive conditions were drawn up for these works and they are referred to briefly later in the Paper.

For the specialized design of the main works the contractor engaged the services of Mr Coyne direct, and no time was lost in submitting a detailed design, together with a draft specification and schedule of quantities, the price, of course, conforming to the offer which had previously been made.

Before the end of February 1953, the contractor was already moving equipment on to the site and on the 1st March, 1953, the work was formally started.

FIG. 6.—CABLES TENSIONED AND RETAINING WALL UNDER CONSTRUCTION

FIG. 7.—COMPLETED STRUCTURE FROM RIGHT FLANK

FIG. 8.—CREST OF SPILLWAY SQUARED OFF AND BRIDGE IN POSITION WITH DRILLS OPERATING ON IT

FIG. 9.—CONGESTION ON SPILLWAY BRIDGE

Design

Design criteria

In the design of a conventional type of dam adequate resistance must be provided against two principal causes of failure:—

(1) *Resistance against overturning.*—The criterion generally adopted to ensure adequate resistance against overturning is that no tension should be developed on the upstream face of the dam. This not only avoids undesirable cracking but also provides an automatic factor of safety against overturning. Sometimes design on the basis of this no-tension criterion is supplemented by direct analysis of vertical pressure under extreme conditions when some cracking is allowed to develop.[5]

(2) *Resistance against sliding.*—In analysing the stability of the structure against horizontal forces which tend to produce sliding it is usually assumed that these forces are resisted by friction alone; sometimes, however, analysis is also made allowing for shear resistance in the concrete in addition to horizontal friction.

The effect of embodying prestressed cables in a gravity dam near its water face is to add at any section a vertical force which, by reducing the overturning moment and by making possible development of greater friction forces to resist sliding, provides additional resistance against the two principal causes of failure referred to earlier. Prestressing, therefore, permits the strengthening of either a dam that requires raising or one that has been under-designed, it being assumed, of course, that the foundations on which the dam is erected are capable of resisting the additional loads.

In designing the raising of the Steenbras Dam it was decided that no tension would be allowed to develop at the upstream face at any section and that the maximum permissible value of the coefficient of friction required for the balancing of the horizontal forces (shear in the concrete being neglected) was to be 0·75.

Uplift pressure was taken as being 0·6 of the full static head at the upstream face of the dam, diminishing to zero at the downstream toe, and uplift was assumed to act over the whole area of the section.

Cable tension

First step in designing prestressed cables for raising the dam was to calculate the total vertical force required for stability at any section. Since the cables run from top to bottom and are designed for the critical section there is an additional factor of safety at other sections.

As is the case with all prestressed concrete structures, allowance must be made for the effects of creep of both steel and concrete. The applied prestress must be made significantly higher than the ultimate tension required. As a result the cable is more heavily stressed during initial tensioning than subsequently; thus the operation of prestressing also provides a valuable test of the strength of the cable and of its anchorage.

Local conditions at Steenbras Dam

The original Steenbras Dam was a gravity concrete structure consisting of a curved portion and two straight flanks, in which precast concrete blocks were used as forms. It was raised in concrete to its present height in the middle 1920s.

Expansion joints were provided on the raised structure generally at 40-ft intervals.

The seal was made by copper strips, but no interlocking concrete keys were used. In calculating the prestress required, therefore, each block between expansion joints on the flanks was considered separately, a sufficient number of cables being provided in each case to ensure stability.

It was assumed, however, that the curved central part of the dam would function as an arch. This assumption was supported by observations in the inspection gallery, since construction joints which intersect the gallery are completely closed and watertight in the central part of the dam, whereas those on either flank show appreciable opening and in some cases slight leakage. The curved part of the dam, acting as an arch, was found to be more than adequately stable, even under the loading developed with a considerable degree of raising, and it was therefore unnecessary to strengthen it.

Cable anchorages: depth

The prestressing cables must be anchored at a depth sufficient to ensure adequate resistance against their pulling out of the rock. Decisions in this regard were made on a semi-empirical basis. It was assumed that the tension in the cable is resisted by the weight of the overlying rock contained in an inverted cone with its apex at the mid-point of the cable anchorage. Where the cables are close together the cones of course overlap, and this must be taken into account in assessing the weight of rock available.

The value assumed for the angle between the sloping face of the cone and its vertical axis is of course critical to the analysis. It depends on the nature of the foundation, and its assessment requires considerable experience and engineering judgement. Mr Coyne, on the basis of observations made during a personal visit of inspection to the Steenbras Dam, decided on a value of 45° for the cone half-angle for this site.

Cable anchorages: length

The length of cable anchorage is also a critical dimension. Once again the character of foundation rock is of considerable importance, and design cannot be based on calculation alone. For cables of the size and spacing finally decided on, it was decided that an anchorage length of 6 ft would provide a sufficiently adequate factor of safety; but in order to give complete assurance a length of 8 ft was adopted. It was further decided that the specification should provide for under-reaming of the holes wherever foundation rock of sub-normal quality rendered it necessary.

Cable size

In designing the cables it was decided to use 0·20-in.-dia. steel wires having an ultimate tensile strength of 213,000 lb/sq. in. and a 0·15% proof stress (as defined in British Standard No. 18 of 1950) not less than 157,000 lb/sq. in.

A series of studies was then made of the economics of a number of alternative designs using various cable sizes and spacings. Among the important factors considered were the costs of drilling, of handling cables, and of forming anchorages for heads. The problems arising from difficulty of access across the narrow top of the existing dam and the limited space available for the cable heads were also borne in mind.

These studies resulted in the adoption of 1·4-in.-dia. cables, each made up of thirty-seven wires of 0·20-in. dia. The minimum diameter of hole to be drilled through the concrete to accommodate the cables was fixed at 2½ in. The calculated

prestress required on each of these cables was 70 tons and to allow for creep the initial tension required was 77 tons.

Cable spacing and arrangement

It will be recalled that the possibility had to be envisaged of a further raising of the Steenbras Dam in the future. In that event the cables now installed would still retain their full value in applying a prestress to the lower sections, but it would be necessary to install a certain number of additional cables to provide additional resistance against the further increase of the forces acting on the dam. Spacing of the prestressing cables was therefore designed to allow room for later installation of additional cables.

Cables were designed on a semi-endless system whereby each would run from anchorage to anchorage over a semi-circular precast concrete head. This rendered tensioning of the cables a simple process. To apply the prestress hydraulic jacks were placed under the heads, which, after the load had been applied, were retained in position by precast concrete packing (See Fig. 6, facing p. 32).

The general layout of the cables is shown in Fig. 1, Plate 1.

Precast heads and packings

Computed stresses in the cable heads were found to be high. The strength of these heads is of great importance, and design was therefore checked by full-scale testing to destruction, undertaken in Paris under the personal supervision of Mr Coyne.

The precast concrete packings, also highly stressed, were heavily reinforced.

Distribution beam

High intensity compression stresses were developed on the dam structure immediately below the packing of the cable head. To spread these stresses at a low intensity to the concrete of the existing structure, a distribution beam was provided. This consists of a layer of high-strength reinforced concrete of 12-in. minimum thickness, except on the spillway, where the whole first lift of concrete up to the stressing level is of high-strength concrete, with reinforcement placed just below the stressing level.

Raised portion of the dam

The raised portion of the dam was designed over its whole length as a cantilevered retaining wall, anchored by reinforcing bars drilled and grouted into the existing mass of concrete.

On the non-spillway section of the dam the cable heads are behind the retaining wall and the cables are completely encased in concrete to obtain protection against weathering as well as for æsthetic purposes (Fig. 7, facing p. 32). On the spillway the heads are embodied in the uppermost lift of concrete forming the spillway crest.

Spillway crest

Design of the spillway crest is shown in Fig. 2 which also gives details of the demolition and rebuilding required.

Provision of a spillway of the conventional type in which the profile follows the nappe would have been difficult. It would have entailed fairly extensive demolition of existing work and the placing of large volumes of new concrete within formwork

that would possibly have been somewhat complicated. It was, therefore, decided to adopt a profile which permitted the nappe to spring clear.

Anti-corrosive treatment

The water of the Steenbras reservoir is typical of a moorland catchment, and since its pH is as low as 5·05 every precaution had to be taken against possible corrosion of the cables.

To reduce the possibility of the acidic water reaching the cables the cable holes were kept as far back as practicable from the water edge and were pressure-grouted before the cables were inserted as well as after tensioning and testing had been completed. The cables were thus entirely sealed within a waterproof cement medium.

As an additional precaution all water in the cable holes was kept alkaline during the period between anchoring the cables and final grouting.

Specification and contract

The contract was drawn up to conform generally to the General Conditions prepared by the Institution of Civil Engineers jointly with the Federation of Civil Engineering Contractors and Association of Consulting Engineers and contained the normal clauses for engineering works. Some special points have already been mentioned, notably the item referring to the cables. The following further clauses of the specification, however, are of particular note:—

Maximum water levels.—Since the contractor was responsible for design, limits of responsibility were set by defining normal full supply level at Reduced Level 1136·75 and top water level at R.L. 1142·25. Design work throughout is based on these figures.

Holes to receive cables.—With the holes arranged closely in groups, the specification laid down a deviation from vertical of not more than 1 in. in 10 ft, and a deviation from a straight line in any 10-ft section of not more than ½ in. The allowable drift from vertical ensures that holes do not cross nor run into each other, and the straightness restriction ensures that there will be no undue friction on a cable.

Before drilling had actually commenced, it was impossible to say exactly what individual anchorage conditions would be like, and an item was therefore included in the schedule to allow for under-reaming any anchorages that were doubtful.

Homing-in of cables.—The specification allowed for two methods of homing-in cables, prescribing the use of neat Portland cement for grout, with a water/cement ratio of not more than 0·50. One method envisaged first placing the cables and then injecting the grout to the bottom of the hole through a tube; the other allowed for first placing the grout at the bottom of a hole, and then working the cable into it.

It was specified that a period of 21 days should elapse after homing-in and before tensioning of cables.

If the cable was anchored at some point higher in the hole than calculated the cone of rock required to develop the tension on the cable would be reduced. The amount of grout placed or injected to the bottom of a hole was therefore required to be accurately measured.

Tensioning of cables.—The total pull required on a pair of cables was 154 tons, and the specification called for the use of 200-ton jacks with metallic safety nuts on their heads which could be kept up with the raises as jacking proceeded. The specification prescribed that, as tensioning proceeded, a continuous record was to be made of load applied, extension, and time intervals.

Since the tensioning of cables is of the essence of the job, extracts from the more important specification clauses in this connexion are quoted hereunder:—

(*Cl.* 21. *Para.* 2.)

"When a jack has extended about $\frac{1}{2}$ inch short of its total lift the packing pieces shall be inserted as shown in the Drawings between the top of the pedestals and the undersides of the heads. As many packings as can be put in with the free hand shall be used. No hammering of packing pieces shall be allowed. The thickest packings shall be placed at the bottom with thinner ones on top. When sufficient packings have been placed evenly under both sides of the head the pressure on the jack shall be gradually released, the cables allowed to contract and the head to rest on the packings. The jack shall then be closed up and packings, specially prepared for the purpose, placed on top of it so as to give the full travel for the second lift. The process shall be repeated until the specified load is reached."

(*Cl.* 2. *Para.* 4.)

"The cables shall be loaded to their full working load of 70 long tons each plus 10 per cent. It will be found that due to settling of the cable, and creep of the steel, the initial load of 77 tons will gradually diminish. The Contractor shall continue to adjust the extension until such a time as the further extension of the cable to obtain the required loading after 1 hour is less than $\frac{1}{4}$ inch. . . ."

(*Cl.* 22. *Para.* 1.)

"When the procedure specified in clause 21 has been completed to the approval of the Engineer, permanent packings shall be put in place between the top of the pedestals and the underside of the head. As many thick packs shall be used as possible followed by the next thickest which may be inserted. The total rise of the head from its starting position shall be accurately measured using calipers or other approved measuring device, and packings selected which total this rise exactly. The packings shall be placed by hand and no hammering or levering will be allowed. In order to facilitate this, the head may be jacked up an additional distance not exceeding $\frac{1}{8}$ inch."

(*Cl.* 23. *Para.* 1.)

"After a period of 28 days from the time of tensioning, a test was called for in which, in terms of this clause, the heads shall be jacked up just clear of the packings and the load ascertained. If this exceeds 70 long tons per cable, the cable shall be deemed satisfactory. If it is less than 70 long tons per cable the head shall again be jacked up until the load becomes 77 tons per cable as described in clause 21, and the cable allowed to stand a further 14 days when it shall again be tested as specified in this clause."

Execution of Work

Construction programme

In the preparation of a construction programme two features were of vital importance, namely:—

(1) Access to work.
(2) Conservation of water in the reservoir.

The difficulties of access are illustrated in Figs 6 (facing p. 32) and 9 (facing p. 33). Since all the materials and plant had to be transported along the top of the dam wall, that going to the left flank having to be taken over the spillway, every operation on the wall had to be so scheduled as to avoid obstructing other work. The access route was never greater than 6 ft in width; mostly it was only 4 ft. Its extreme length was 1,400 ft.

To provide a working platform for drilling, and a square base for tensioning, the rounded surface of the existing spillway had to be removed, and for this purpose it was necessary to demolish the top of the spillway to a depth of 4 ft 9 in. below the old crest. If loss of water was to be avoided, therefore, this demolition and rebuilding to previous crest level had to be done at a period of low reservoir levels. A study of recorded reservoir levels showed that the period had, consequently, to be limited to the months January to May.

It was plainly impossible to complete the entire work of demolition, drilling, installing cables, stressing, and finally rebuilding to the new crest level within this restricted period of one summer. The programme of work therefore called for demolition and rebuilding of the spillway to its old crest level before May 1953, drilling from a bridge throughout the winter of 1953, and completion of the work during the summer of 1954. (See Fig. 8, facing p. 33.)

The need to conserve water thus dictated the construction programme for the spillway, and in view of the difficulties created by the restricted access available, it was decided to confine work at the start of the job in March 1953, to a limited amount of drilling until the spillway crest could be demolished and rebuilt on the new section to its original level.

The resulting programme of work is shown in Fig. 3, Plate 2.

Demolition of spillway crest

This work proceeded normally and satisfactorily until heavy rains fell in May 1953. The temporary spillway bridge had been designed to support flashboarding, so that in the event of a sudden rise of water level, a temporary closure could be made of any gap, and on the 30th May this procedure was adopted. With about 30 ft of old spillway remaining undemolished, work on this section had to be suspended until January 1954.

Drilling

It had been intended to use percussion drills for the bulk of the work, and diamond drills only to ascertain rock levels and to take samples of rock and concrete.

Almost from the outset, however, difficulty was experienced in keeping holes within the permissible tolerances, particularly so with percussion drills.

The site being exposed to frequent winds, it was not possible to rely on the use of a plumb bob for setting up holes to drill sufficiently close to vertical. In starting, therefore, holes were drilled approximately vertical but to a larger diameter than required, and casings inserted and carefully plumbed, using a level fitting specially made for the purpose. These casings, 5 ft in length, acted as guides to start drilling.

Unless cutting tools (either percussion or diamond) were followed by centralizers only $\frac{1}{8}$ in. smaller than the bits, holes were found to deviate both in the concrete and the rock, and the allowable tolerance could be attained only with percussion drills on small holes, which were thereafter reamed out. The procedure of drilling small percussion holes initially had the advantage that in the event of a hole drifting, it could be grouted up and redrilled with diamonds.

The Table Mountain sandstone, on which the dam is built, dips steeply and consists, in the Steenbras area, of extremely hard layers interbedded with softer material. This characteristic rendered percussion drills useless since these tended to follow the dip down the hard layers, and compelled the use of diamond drills to keep holes within the specified tolerances. Another difficulty arose from the fact that the original concrete structure contained pieces of wood, bits of reinforcement, wire shuttering ties, and other obstacles that seriously interfered with drilling to tolerance. In many cases holes had to be drilled a number of times before they would pass through an obstruction without a kink; in a few cases holes had to be re-sited.

Nevertheless only one hole was completed outside the specified tolerances, and fortunately that one deviated away from others in the same group. The obstruction which caused its drift was a steel rail embedded in the concrete.

The delays engendered by these difficulties caused drilling to extend over a longer period than that originally planned, but by careful and timely reorganization, drilling was kept ahead of other work.

Grouting of holes

With a few exceptions holes were watertight in the concrete and at the rock contact, but showed heavy leakages at 10 ft to 30 ft into the rock. Usually leakage occurred in soft layers characterized by iron stains, which were found below hard quartzitic bands (the presence of which was indicated by slow drilling). Some of the leakages exceeded the pumping capacity of 1,500 g.p.h., and to contend with this the amount of grouting was increased. Pressure was applied up to a maximum of 200 lb/sq. in., and to avoid dangerous uplift on the wall, relief points were arranged at groups of holes adjacent to those being grouted. Grouting was continued until leakage was reduced to less than $1\frac{1}{2}$ g.p.h. measured as a drop in the free water surface of a hole.

On one limited section of the wall, a discharge of brown mud was noted from release holes adjacent to holes under treatment. Drilling in the area had not indicated any mud seams, and it therefore appeared likely that pressure grouting had found a weak spot and was squeezing out the soft material. This occurrence led to a careful check for any other weak spots; additional work entailed, however, caused a serious delay in the placing of cables, particularly in the section of wall adjacent to the spillway.

Cable anchorages

The fact that the rock underlying the dam, even though very solid in the main, could not be drilled by percussion methods necessitated reconsideration of the question of cable anchorages. Mr Coyne's experience had shown that a hole drilled by percussion methods offered an excellent grip; but he was not completely satisfied with regard to diamond drilling in view of the smoothness of the surface of the samples of diamond-drilled core submitted to him.

The following alternative proposals were therefore suggested:—

(1) To drill anchorages by percussion tools, but to relax all tolerances regarding straightness of holes over the anchorage length (9 ft from the bottom of the hole).

(2) To complete holes with diamond drills, and to under-ream the bottom 9 ft using a special diamond tool, giving a step of $\frac{1}{4}$ in. and a straight-sided hole.

(3) To complete holes by diamond drilling, and to under-ream the bottom 9 ft by means of a tool leaving an irregular, wavy surface.

Mr Coyne decided on a percussion-drilled anchorage and asked that a field test of this type of anchorage be carried out.

Test cable

The test cable was sited at the end of the dam wall. The top of the anchorage was at a depth of 22 ft, of which 20 ft was in rock, and anchorage length was the specified 8 ft. Almost the full length of the two holes was below water level so that conditions approximated to those likely to be encountered normally.

The various components used were selected at random from stock, and the test therefore applied not only to the anchorage but to all components, including the first piece of cable manufactured.

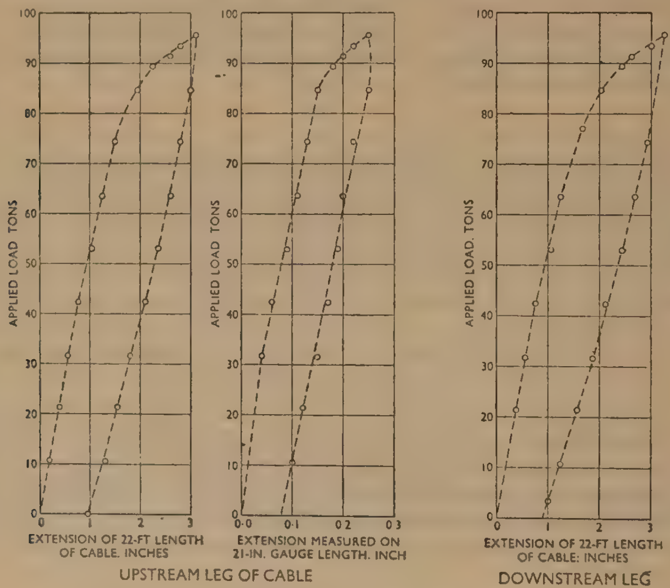


FIG. 4.—LOAD/EXTENSIONS GRAPH FOR TEST CABLE

Load was increased in 500-lb/sq. in. stages on the pressure gauge to 190·8 tons which is a considerable overload on the design figure of 154 tons. Measurements of extensions were taken over the whole 22-ft length of each leg of the cable, and the results recorded on a load-strain graph (see Fig. 4). The strain over a 21-in. gauge length was also recorded. This corresponded closely with the strain measured over the whole length of cable to the top of the anchorage, thus indicating that no movement took place in the anchorages.

The above results being considered satisfactory, anchorage drilling commenced immediately after the test.

Homing-in the cables

Using overseas practice as a guide, experiments in homing-in a cable were conducted in the contractor's works in Johannesburg. A hole was drilled to 100 ft and cased with steel tubing. Inside this a removable 2½-in. tubing was set, thus simulating holes drilled in the Steenbras Dam.

Since the reservoir would be full when cables were being homed in, it was possible that there would be some water in most holes. The 2½-in. test hole (tube) was accordingly filled with water.

A series of anchorages were made and the 2½-in. tube was extracted and cut open for examination. The tests covered the following:—

(1) Study of alternative methods of homing-in a cable (pre- or post-grout placing) and study of pumping grout of various consistencies.
(2) Comparison of various methods of placing grout.
(3) Comparison of the use of various types of cement.
(4) Study of modifications on selected method of placing grout.

(1) It soon became apparent that grout would have to be placed before the cable. The maximum size of tube that could be placed in the hole before the cable seemed to be about ½ in., and great difficulty was experienced in pumping grout of the required water/cement ratio of 0·50 through a tube this size. Although pumping was possible even with a water/cement ratio as low as 0·38, the slightest hesitation of the pump, or irregularity, produced a blockage.

If the cable was placed before the tube, however, a larger-diameter (⅜-in.) tube could be worked down to the top of the anchorage but not below this point. Grout pumped down the larger tube had to fall the last 8 ft through water and thus became very diluted and useless for producing a satisfactory anchorage.

(2) The contractor's experience in cementing deep diamond-drill holes had shown that grout could be placed by means of a container fitted with a pressure-operated valve at the bottom. After a container thus fitted has been lowered, the application of water pressure to the grout surface opens the valve and grout flows out until pumping ceases. The container is raised as the grout discharges.

The method has the drawback, however, that turbulence is liable to dilute the grout unless the container is raised carefully. Furthermore, a grout thicker than about 0·50 water/cement ratio cannot be handled, for the transmission of pressure to the foot valve under such conditions becomes doubtful.

To overcome these difficulties it was decided to try a plain open-ended tube blocked at the bottom with only a glass disk. An electric detonator was placed just above the glass, and the tube already filled with grout lowered to the bottom of the trial hole. When in place the detonator was fired, and the tube then slowly withdrawn.

The results were encouraging, but not entirely satisfactory. Even though the glass was completely shattered and there was no resistance to the grout discharging, a considerable dilution of the top of the grout charge was observed. A further complication arose from the fact that once the water/cement ratio fell below 0·30 it was extremely difficult, if not impossible, to get grout into the placer without air-locking.

In the light of the experience gained, the method of placing was once again modified and the following procedure evolved: to avoid airlocks the grout was filled into a 6-in. cylinder, from which it was discharged by a piston into a placer, 2 in. int. dia. × 13 ft long. It thus became possible to use a much thicker grout, of water/cement ratio 0·25. The placer tube was fitted with a piston which could be driven down by air pressure, so extruding the grout at the bottom. To eliminate turbulence the placer tube was fitted at its lower end with a 9-ft-long nozzle of 1 in. ext. dia. and a foot valve which opened only on contact with the bottom of a hole (see Fig. 5).

Table 3 shows the water/cement ratio at various depths of trial anchorages as determined by analysis after recovery.

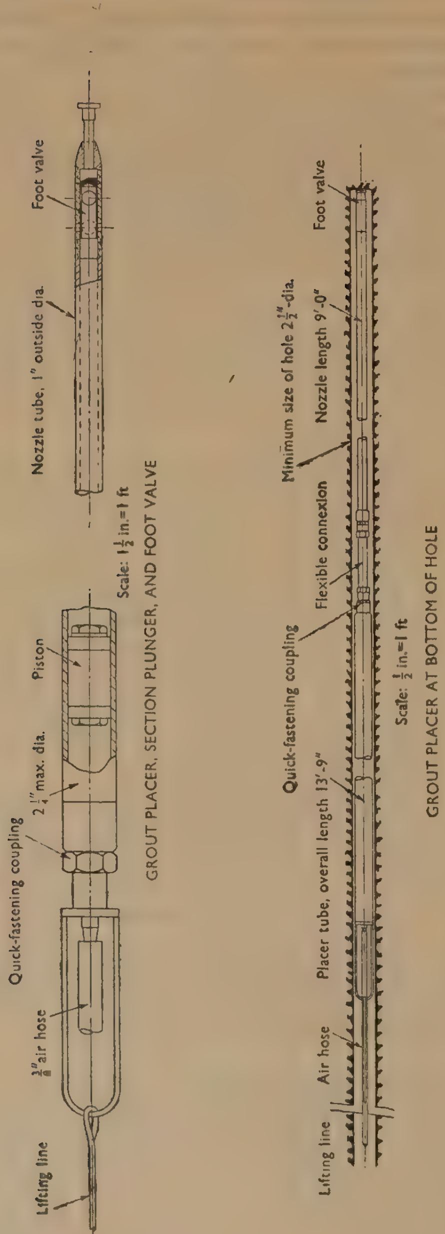


FIG. 5.—GROUT PLACER

TABLE 3.—WATER/CEMENT RATIOS ACHIEVED IN TRIAL ANCHORAGES

Type of cement, placer, and water/cement ratio	Distance of cut from bottom of anchor brush: ft	Water/cement ratio
Portland cement—Brand 1. Explosive placer 0·50	Test a failure, grout very dilute	—
Aluminous cement Explosive placer 0·30	0·5	0·37
	2·5	0·33
	4·5	0·24
	5·9	0·37
	6·9	0·44
	7·8	0·85
Portland cement—Brand 2. Explosive placer 0·30	0·5	0·31
	2·5	0·30
	4·5	0·38
	5·5	0·33
	6·5	0·63
	7·5	0·70
	8·5	0·98
Portland cement—Brand 3. Explosiver placer 0·38	0·5	0·34
	3·5	0·35
	4·5	0·30
	5·5	0·35
	6·5	0·46
	7·5	1·38
	8·5	1·88
	9·5	—
Portland cement—Brand 1. Pressure placer 0·30	0·5–6·5	Obviously excellent; no analysis made.
	7·5	0·33
	8·0	0·42
	8·5	1·00

The results show the deleterious effect of turbulence on the top of anchorages, and that this is aggravated by increase in water/cement ratio. They also demonstrate that the method of placing adopted is satisfactory.

(3) The effects of turbulence and the settlement of heavier particles of cement were also studied by simulating anchorage conditions in a 2½-in. pipe on the surface. Tappings were taken at various points and water/cement ratios then determined.

The conclusions reached may be summarized as follows:—

(*a*) Placers must be extracted slowly and cables inserted slowly and under control.

(*b*) Cements of larger grain size are better than finely ground cements. The critical characteristics of the cements tried are shown in Table 4.

In the experimental work all mixes were carefully proportioned by weight. On the site, however, it was found impossible to control the water/cement ratio in this way, for the work was done in midsummer and evaporation seriously affected the

TABLE 4.—PROPERTIES OF VARIOUS CEMENTS

Type of cement	Surface area Sq. cm/gm cement	Percentage retained on 170-mesh screen
Portland cement—Brand 1	3,600	4·0
Portland cement—Brand 2	2,560	8·8
Aluminous cement	2,188	4·4
Portland cement—Brand 3	3,000	4·5

results. A type of viscometer was therefore made on the job and calibrated against known mixes in the cool of the evening, and this was used throughout the work for adjusting the grout mix immediately before it was placed.

Preparing and handling cables

Each cable length was made up complete in the manufacturer's works, with brushes, bindings, and shoes, and marked with a metal tag giving its number on the wall and exact length. Delivery was arranged to suit the construction programme; and cables were homed-in as soon as possible after arrival. They were fed into the holes by hand one end at a time, without any notable difficulties (Figs 10 and 11, facing p. 48). On some of the longer lengths there was a tendency for cables to run away as the weight in the hole increased. To overcome this tendency a mechanical braking device was evolved, but since the job was then nearing its end it was never used.

Tensioning of cables (Fig. 12, facing p. 49)

The procedure laid down in the specification was closely followed and proved completely satisfactory. Indeed the only point calling for comment was that the specified test 28 days after tensioning seemed unnecessary. All 28-day tests showed residual tensions above the designed figure of 70 tons. On the other hand the waiting period of 28 days delayed construction and left the cable exposed to possible corrosion for that much longer.

One other point of interest was that pressure gauges were tested before each day's tensioning on a dead-weight pressure-gauge tester set up on the job.

Cable anchorage failures

Cables were tensioned using the method described in the section headed "Design," and no failures occurred until the first week of March 1954, by which time a great many had already been installed. Between that date and the application of measures applied to counteract the deficiencies disclosed there were six failures of the anchorages of single legs of cables, out of a total of 326 on the job.

Extensive enquiry followed the first failure, the causes of which were revealed by examination of the extracted cable and which were generally attributable to grout dilution. All failures occurred on long cables which had been allowed to run into the grout too rapidly; a fault in construction subsequently admitted by the placing crew.

A further and perhaps equally important cause of failure was excessive enlargement of the anchorages in drilling, with consequent reduction in anchorage length. Evidence of this was the clear "high water mark" of grout at a point on these cables well below the designed length of anchorage.

One failure was particularly interesting. It occurred at 154 tons—the full design load, the length of good grout on the cable being 3 ft 4 in. The minimum length of anchorage required was thus made apparent.

All anchorages, it will be recalled, were percussion drilled with relaxed straightness tolerances, and there is little doubt that many deviated very considerably. To check this, one hole was re-drilled with diamonds, and a deviation in the anchorage of 2 in. in 15 in. was discovered, or 7½°. Where a hole deviated, the alternating hard and soft layers which cause deviation would also cause enlargement. Once the causes of failure had been established immediate steps were taken to eliminate all weaknesses in placing, and as a result no further failures occurred.

Protection and final grouting of cables

Experiments during construction showed that once a hole had been pressure grouted there was sufficient residual alkalinity to neutralize the acid water in the hole, and no protection of cables was therefore necessary once they had been homed-in. For the final grouting ¼-in. steel tubing, flush-jointed in 10-ft lengths, could be worked down to the top of anchorage grout with ease. It was thus possible to flush holes out thoroughly before finally grouting up, and then to fill up from the bottom, displacing the water. The grout used thus for filling had a water/cement ratio of 0·65.

Prior to the start of final grouting, holes were blocked with a neat-cement plug through which a ¾-in. tube was inserted. The ¼-in. tubing earlier referred to was passed through the ¾-in. tube and grout was pumped until all water had been displaced. The final seal was made by injecting cement at 200 lb/sq. in. through the plug after withdrawal of the ¼-in. tube. During this process considerable leakage of grout occurred through the strands of the cable, giving positive assurance that the final seal was completely watertight.

General concrete work

There is little of special note in the general concrete work, which followed normal practice. The amount of work entailed was not large enough to warrant special handling arrangements, and its main feature was that all operations had to be devised to avoid blocking the narrow access way.

Emergency Storage of Water

Construction work proceeded normally and at schedule speed throughout the summer of 1954; but it was never possible to make up the time lost by the drilling difficulties, the extra grouting of underlying rock required, and the early rains of May 1953.

The summer of 1954 was exceptionally dry and the water level in the reservoir sank to 30 ft below the old spillway crest, equivalent to a storage of only 1,400 million gallons as compared with a total capacity of 5,991 million gallons. At that juncture, the 22nd April, 1954, it seemed that, with the work only about a month behind schedule, there would be little or no trouble in raising the spillway crest in time to store the winter rains.

But a rapid and dramatic change in the weather was about to transform a satisfactory situation into a well-nigh critical one. In April, normally an early autumn month, one of the worst and wildest winters at the Cape for many years set in, and the Steenbras reservoir filled rapidly. By the 17th May, 1954, the concrete on the

spillway had been raised to R.L. 1134·75, that is, 2 ft below new crest level. At this stage, with the rapidly rising water level, it would have been folly to raise the spillway any higher, for a sudden extra rise would have drowned out work then proceeding on raising the valve tower, and would have outflanked the dam where work was still proceeding on the lower lifts of the flank walls. Along the wall itself, the distribution beam top was about 3 in. above the designed spillway crest, so that there was no danger there, although a rise of water to that level would have seriously upset construction of the main retaining wall, which was then about half-finished to R.L. 1142·50. (See Fig. 6, facing p. 32). By the 30th June the water level had risen to R.L. 1131, nearly 1 ft above the original spillway crest level, and heavy rains were continuing. The flank walls were safe at R.L. 1137 or above, and the retaining wall was within a week of completion to R.L. 1142·50. All preparations had been made to continue the spillway, then 2 ft below the new crest level, to full height, but with the rapid rise of the water it became doubtful whether the concrete could be placed in time.

At this stage the City Engineer's Department stepped in, and by co-operation between its men and the contractors, a brick wall was carried across the spillway crest, raising it to full supply level. Even this could not be done in time, and water overflowed on the 9th July. However, brickwork was continued when the weather allowed, and in a close race against time was completed to crest level on the 19th August, 1954. By the 23rd August, the reservoir was overflowing at the new designed crest level, albeit over only a temporary spillway.

Ancillary works

The two major items of ancillary works were the raising of the valve tower and its access bridge and the construction of flank walls to the main dam. These two works were let as separate contracts in March 1954, and work on both started almost immediately.

The raising of the valve tower presented few problems of note. The associated raising of its access bridge was undertaken by jacking up the existing bridge to its new position and constructing an additional span.

The construction of the right flank wall was an easy matter. On the left flank however, difficulties of access had to be faced. The attendant problems were neatly solved by the use of the "Colcrete" process.

Close to the end of the wall there were the remains of an old quarry, which provided a ready source of good stone of fairly large sizes, suitable for pre-packing dry in formwork. Furthermore, there was also a beach nearby of sand rather too uniform in size for normal use, but ideal for Colcrete work. Having these two materials adjacent to the work, the access problem was then reduced to transporting cement along the wall.

Formwork was erected in the normal way and filled with stone packed in at random, except that flat stone surfaces were kept away from the sides of forms (Fig. 13, facing p. 49). As packing of stone proceeded, 5-ft lengths of scrap 3-in. piping were placed vertically in the aggregate, spaced at about 5-ft centres. When the form was ready and filled, sand-cement grout, containing an additive for increasing workability, was pumped from the mixing point on the beach direct through a flexible hose into these vertical 3-in. pipes and the grout thus filled up the interstices in the stone from the bottom. The resulting concrete and its finish against the formwork were completely satisfactory.

Conclusion

The processes utilized in strengthening and raising the Steenbras Dam were, so far as methods of dam construction are concerned, entirely new to South Africa.

For a public official, and indeed a public authority, to embark upon a new process is not an easy matter.

Very rightly it is not for an official or local authority entrusted with the control of public funds to allow experimentation on methods unproven, at the expense of the public. On the other hand fear of trespassing beyond the confined barriers of established practice should not deter a conscientious officer from recommending processes which objective and scientific investigation have shown to be sound and reliable, and whose application would save the public considerable expenditure.

In this instance, however, it was not only the direct financial savings which were significant in determining the policy to be pursued, no matter how important such savings might have been. Very practical considerations—expedition, the possibility of avoiding restrictions which could have had serious repercussions on the City's development, the avoidance of almost irreparable damage to one of the City's most valuable scenic attractions—all these, combined with the possibility of saving about £200,000 compared with the cost of traditional methods, were held sufficient to justify a departure from normal customary methods. This departure, moreover, applied not only to the methods of actual construction but also to the manner in which the contract was to be awarded.

The desirability of competitive tenders for public works is too well known to need elaboration. There are times, however, where this normally satisfactory method of executing public works may lead to undesirable complications. Where highly specialized processes and operations are to be carried out it is an essential prerequisite that whoever is charged with carrying out the work shall be completely qualified to do so, by virtue of experience, competence, and skill.

In such highly specialized undertakings the lowest tenderer need not, and very often certainly will not, in the long run, prove the cheapest.

From the financial point of view the project can be considered eminently successful. The total cost to the City Council of carrying out all the works totalled £110,000. By raising the height of the dam wall an additional 1,552 million gallons of water were stored in the first winter rains. The value of this additional water to the City at the statutory charge of 2*s* 3*d* per 1,000 gallons was £174,600 and the sale of this water (which without the raising would not of course have been available) during the summer 1954/55, more than covered the cost of the project. Furthermore there will be a benefit to the ratepayers in perpetuity resulting from the increased assured yield now available from the reservoir.

Apart from the material benefits described above, however, the project and its method of execution demonstrated how effective and useful can be the collaboration which is possible between departmental officers and contractors' employees when all parties concerned are imbued with zeal, enthusiasm, and determination to produce the best possible job at the lowest possible cost.

The work has also opened up new possibilities in the realm of dam construction and rehabilitation; perhaps further study may even show that the methods utilized may be effectively exploited in the design of dams *de novo*

Acknowledgements

In conclusion the Authors gladly acknowledge the work done by, among many others, the Board of Engineers; Messrs D. A. North, J. G. Gräbe, and R. T. Shaw of the Cementation Company (Africa) (Pty), Ltd; Messrs C. Grundy and P. I. Parker of the parent Company; and Messrs D. C. MacKellar, J. G. Welsh, B. D. Kark, and D. Lipsett of the City Engineer's Department, all of whom participated in various stages and degrees in the execution of the project.

References

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2. L. A. Mackenzie, "A Note on Flood Peak Probabilities in certain Rivers of the Union of South Africa." Professional Paper No. 10, Irrigation Dept, Union of S.A., 1947.
3. L. A. Mackenzie, "Flood Peak Probabilities in Certain Rivers of the Union of South Africa." Professional Paper No. 17. Irrigation Dept, Union of S.A., 1951.
4. K. B. Keener, "Uplift Pressures in Concrete Dams." Proc. Amer. Soc. Civ. Engrs, June 1950. Separate No. 25.
5. "Design Criteria for Concrete Gravity and Arch Dams." U.S. Bureau of Reclamation, Nov. 1950.

The Paper, which was received on the 8th June, 1955, is accompanied by twenty-six photographs and five sheets of drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

Correspondence on the foregoing Paper should be forwarded to reach the Institution before the 15th January, 1956. Contributions should not exceed about 1,200 words.—Sec.

Discussion

Mr Morris introduced the Paper with the aid of a film showing the operations described in the Paper.

The Chairman said that when Mr Lloyd Davies had described the dam (reference 1, above) there had been criticism of the estimate of flood discharge, but Mr Morris had provided for 17,000 cusecs, which was about 2½ times the normal maximum flood according to the Floods Committee Report. Quite rightly, he was on the side of safety.

Those who might have to contemplate similar work, or even the construction of a dam from the very beginning on the same principle, would find the Paper extremely useful. The Chairman said that his firm were concerned with a project for heightening an important dam in the Midlands by about 45 ft, about 25% of the present height above the foundation. In the case of Steenbras, the heightening was 6 to 7%, which was much more modest.

It was hoped in the Midlands to have a trial run with a single borehole right through the dam and 120 to 130 ft into the underlying rock, to determine the rock conditions, which were something like those described by the Authors. Then they wanted to satisfy themselves about the arrangements for stressing. The Author had given much food for thought

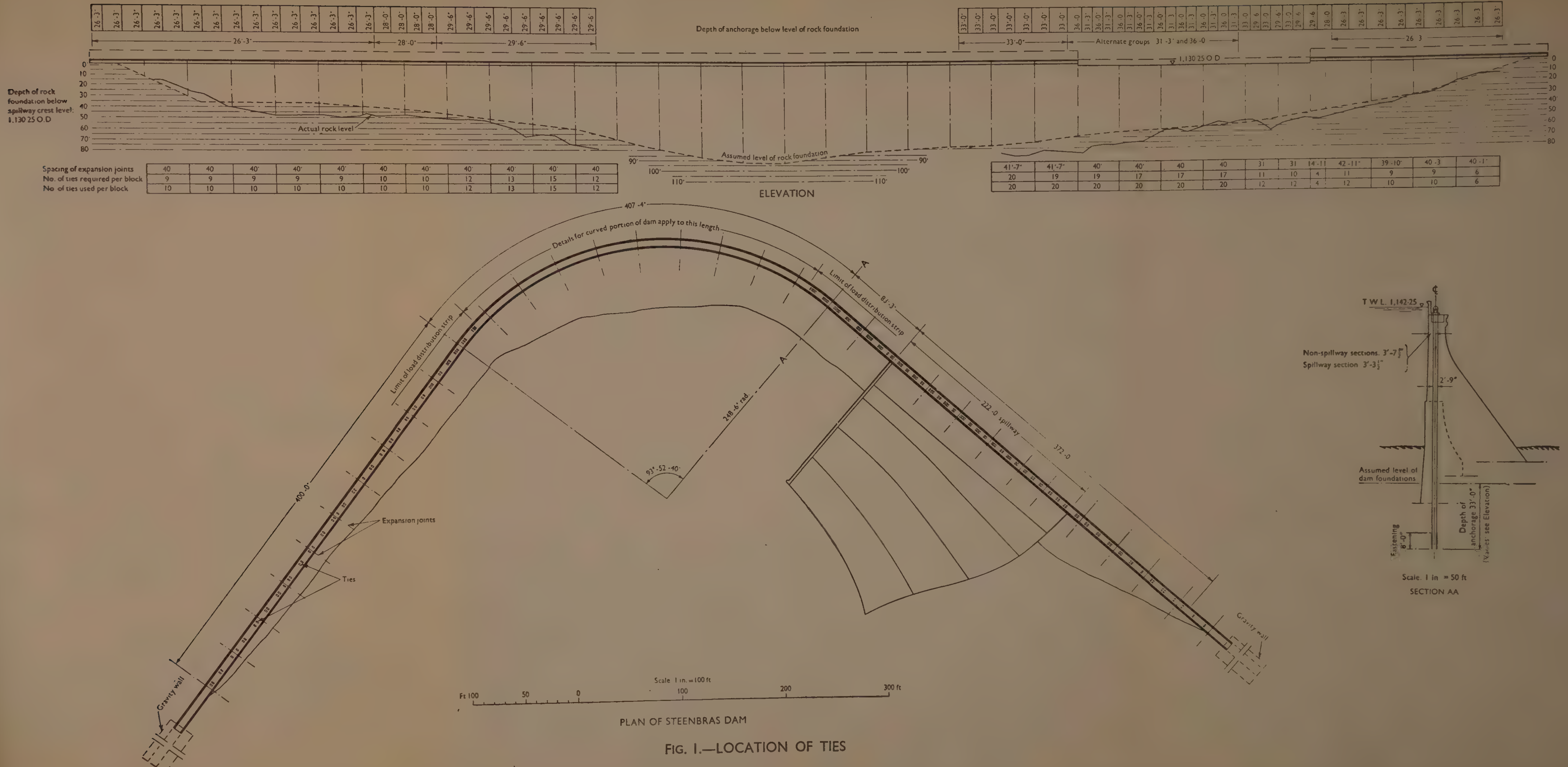
FIG. 10.—CABLE BRUSH ENTERING A HOLE

FIG. 11.—VIBRATING A CABLE AFTER HOMING IN

Fig. 12.—Cable being tensioned; completed cable in background

Fig. 13.—Placing "colgrout" on left flank. Coarse aggregate in place

THE RAISING AND STRENGTHENING OF THE STEENBRAS DAM



Spacing of expansion joints	40	40	40'	40'	40'	40	40'	40'	40'	40'	40
No. of ties required per block	9	9	9	9	9	10	10	12	13	15	12
No. of ties used per block	10	10	10	10	10	10	10	12	13	15	12

41'-7"	41'-7"	40'	40'	40	40	31	31	14'-11	42'-11"	39'-10"	40'-3	40'-1"
20	19	19	17	17	17	11	10	4	11	9	9	6
20	20	20	20	20	20	12	12	4	12	10	10	6

FIG. 1.—LOCATION OF TIES

S. S. MORRIS and W. S. GARRETT

WILLIAM CLOWES & SONS, LIMITED: LONDON.

THE RAISING AND STRENGTHENING OF THE STEENBRAS DAM

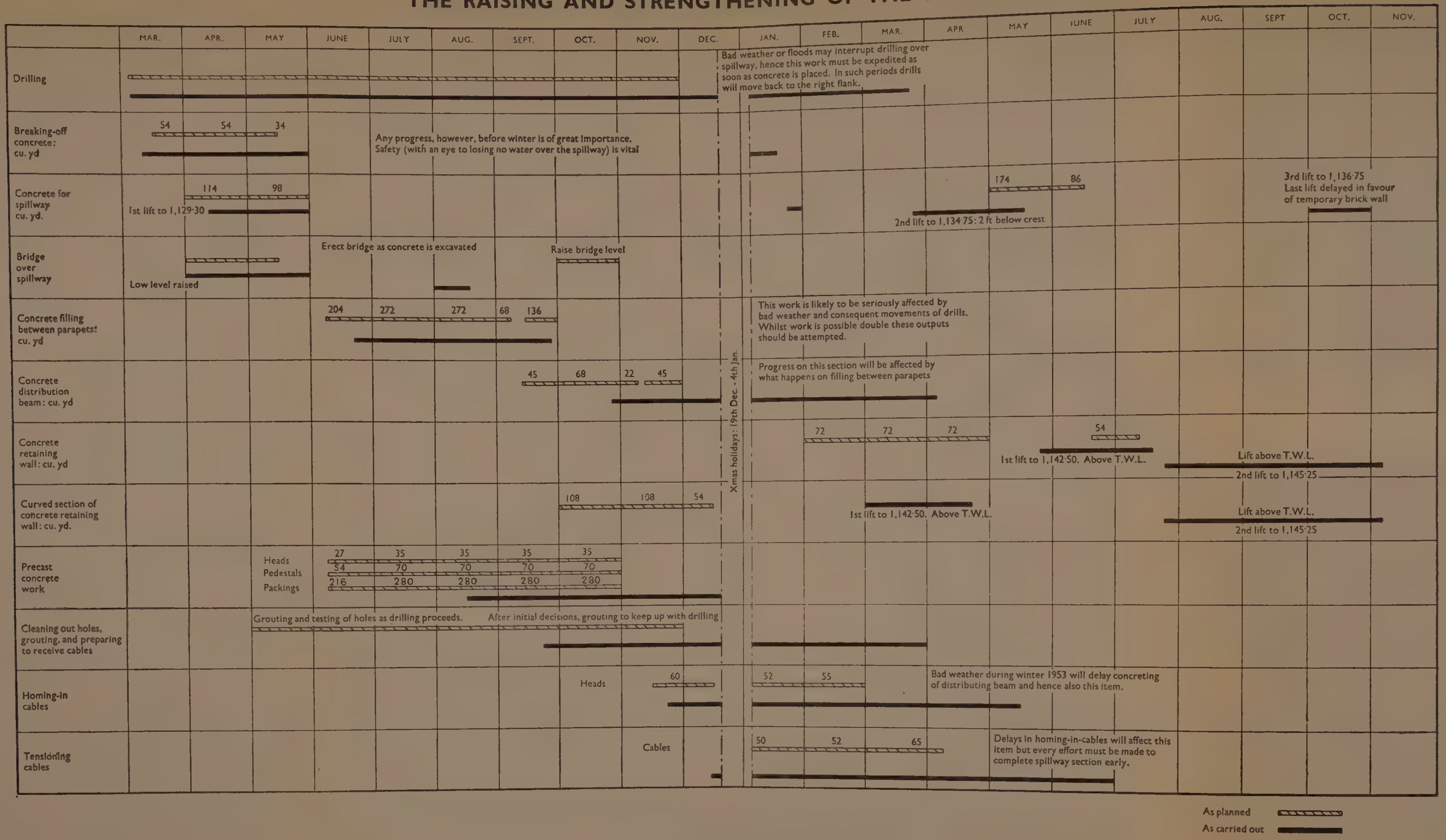


FIG. 3.—PROGRAMME OF WORKS

and his information together with the experience gained in the Midlands would be very useful. If the trial borehole and cable were successful, there would be 200 such boreholes and cables. The job was on a much larger scale than that described by the Authors.

The method described in the Paper had been adopted on the Continent and in North Africa, and a dam had also been designed from the start on the basis of pre-stressing.

Mr R. C. S. Walters (Partner in the firm of Herbert Lapworth Partners, Consulting Engineers) agreed that the Authors had made flood provision 2½ times that envisaged by the Floods Committee. The Floods Committee had in mind a normal flood, which had actually been recorded, and possibly the Author intended his flood to be a catastrophic flood, twice that envisaged by the Institution Committee. That being so, it struck him as interesting that the floods in South Africa were comparable with those in the United Kingdom. Floods in America for similar upland areas might also be similar to the Institution Floods Committee's standards.

He was particularly interested in the provocative remarks in the concluding paragraph of the Paper dealing with the incorporation of siphons or prestressing in a new dam. There was a difference between adding something to an existing dam and incorporating an extra height of dam in a new design; when impounding the equivalent or balancing about 70% of the average run-off, it might not normally be economical to go any further. But if about 1,500 million gallons could be added to the storage for an expenditure of £100,000 on an existing dam, it seemed extremely good going. He had recently been associated with a problem of putting a siphon on an existing dam to cope with about 4,000 cusecs for an expenditure of £20,000, to increase the storage by about 200 million gallons. It was well worth while.

But it had to be ensured that the siphons were *safe* and that the prestressing was *permanent*. The Authors had suggested that they had rejected the idea of siphons because of the difficulties. Mr Walters had visited France to see a comparable siphon (for the same quantity—4,000 cusecs) in operation in a hydro-electric station where the turbines could be shut off for testing the siphon visually.

According to pp. 30 and 31, the Authors were nervous of the siphon because of wave action. Mr Walters had avoided that by making a series of sixteen siphons at different levels. The criterion was that the catastrophic flood level in the area could not be increased. The priming and depriming difficulties referred by the Authors could be overcome where necessary.

Les Cheurfas dam in Algeria had been raised in 1935 on the Coyne system. The total tension in the cables was 1,000 tons. In 3 years that had deteriorated by 40 tons, in the next three years by 4 tons and in the next 3 years by 1 ton. Apparently at Steenbras there was now no means of measuring the tension in the cables, and it may be that Mr Coyne did not deem it necessary to measure it and to go on taking the tension stresses, especially since Les Cheurfas dam, still standing after 20 years, had proved to be perfectly safe. Presumably the cables at Les Cheurfas had been put in holes and grouted up. Had the Authors any means of observing any loss of tension in the cables?

Another Coyne variation Mr Walters had seen* in France was at the Castillon dam. There had been trouble with the abutments and the rock had been reinforced with pre-stressed cables let into the rock. It seemed that prestressing was a safe process and although it was not popular with some Canadian engineers, the French were apparently quite confident about it.

Mr C. M. Roberts (Partner in the firm of Sir William Halcrow & Partners, Consulting Engineers) said that the narrow crest of the dam was one of the chief drawbacks of the method when applied to raising and, in addition to making the work difficult to execute, set a practical limit to the amount of raising. The Chairman had spoken of a 45-ft heightening elsewhere and the Authors had stated that investigations had been made

* Thanks to the kindness of M. A. Decelle, Electricité de France, Marseilles.

into raising the Steenbras dam between 3 and 30 ft. Were the heights approaching 30 ft to be accomplished by the same method as that described in the Paper? Possibly some of the investigations were on the basis of extending the cables up through the raised portion of the dam.

The arched part of the dam was not prestressed; was the arching effect alone the criterion there, or had the Author taken into account arching and gravity in the original portion of the dam and arching and cantilever for the raised portion?

The Authors had referred to the low pH-value of the water, which was surely very aggressive. Apparently a piece of exposed steel did not show signs of deterioration, but it would be interesting to hear a little about the effects of the water on the concrete itself. Classes of concrete were mentioned in the diagrams; but what were the cement content and the nature of the cement in the concrete between the water face and the cables? Was there any protective treatment of the water face?

Mullardoch dam in Scotland had been partly raised by 20 ft in 1951, as described in a recent Paper [6] presented to the Institution. The method used for strengthening had been to thicken the original section. If the saving by using cables was as great as the Authors had said on p. 47, it might be asked why the method had not been applied at Mullardoch. The reason was very simple: only some of the blocks of the Mullardoch dam had had to be raised during construction of the remainder. Various blocks surrounding those being raised were being built to the full required height and the foundations had been prepared for the final height of the dam throughout. The fact that certain blocks had been finished to a lower section was purely a matter of temporary economy, but financial relief had materialized before the completion of construction and it had been possible to raise the comparatively few blocks which had been built to the lower section. Ultimately those lower blocks were required to conform to the same profile as the full-sized ones to meet spillway requirements and there had been no question of adopting a thinner section in the case of the Mullardoch dam.

Mr A. J. Harris (Consulting Engineer) said that the method which the Authors had used with such signal success for this dam had first been developed by Coyne about 20 years previously, and although the techniques of prestressing had developed considerably since then, he thought it true to say that Coyne's cable had not developed very much. There might be a very good reason for that; it worked very well and in all likelihood the cost of the cables was not very great in comparison with the cost of the remaining works.

Nevertheless, certain features of the cable invited comment. The stressing cables were of laid wires and passed over quite a sharp curve over the head of the jacking device, which did not seem very judicious, since the state of stress, which was high to start with, would be considerably complicated by the additional curvature.

Again, one of the advantages of using prestressing as distinct from reinforcing, in a dam such as the Steenbras, was that it was possible to ensure a very high degree of protection of the cable. Had the Authors considered impregnating the cables with bitumen prior to dropping them down the holes, which would provide a very high degree of protection? The creation of bond was of no value, because if the dam ever reached the point where it cracked, the operation had failed and variations in the reserve of strength were a matter of indifference. Bitumen coating would permit the engineers to keep their cables under observation over a longer period; a cable without any protection at all had a very high rate of corrosion, but the use of bitumen would enable the engineers to keep it open, possibly indefinitely.

The Authors had given very little information about concrete strength in either the old or the new dam, and it would also have been interesting to see the distribution of stresses. Stabilizing by prestressing of the type described usually provided a very much more favourable distribution of stresses and frequently a smaller maximum stress, and it would be interesting to have some information on that.

[6] C. M. Roberts, "Special Features of the Affric Hydro-Electric Scheme (Scotland)." Proc. Instn Civ. Engrs, vol. 2, p. 520 (Sept. 1953).

What technique had been used for blocking-up the jacking heads when the cable had been extended?

Most of his comments seemed to have been "Why did not the Authors do this?" It was sufficient to say, however, that the responsibility taken by an engineer in making a dam of such a magnitude with a technique which, while not novel, was bold, was very considerable, and the Authors had every reason to be proud of the result.

Mr Jack Widdowson (of Civil Engineering Department, The Cementation Company Ltd, Contractors) observed that it had apparently been the Authors' original intention to do most of the drilling by percussion methods but that intention had to be abandoned and quite a proportion of the drilling was carried out by diamond methods. Even with both methods of drilling they had had considerable difficulty in maintaining at all times the necessary degree of straightness and verticality tolerances.

He compared the drilling operations at Steenbras with a project where the drilling operations were basically the same—a dam on the Coyne principle, involving the principles of strengthening but not of raising. He referred to the Tansa dam, in India, which was one of the most important features of the Bombay Municipality Water Undertaking. At Tansa the dam was approximately 9,000 ft long and the strengthening operations involved the drilling of 2,400 holes along the top of the dam and the spillway. Each of those holes was carrying a 70-ton cable and was of 2½-in. dia. The average depth was 100 ft and the maximum depth was about 180 ft. Drilling, in the core of the dam, was through rubble masonry of Dekkan Trap basalt in lime-cement mortar. The underlying rock below the dam was also Dekkan Trap basalt. On the whole of the project, all the 2½-in.-dia. holes had been drilled by percussion methods, with no diamond drilling whatever. In the early stages of the drilling operations and after the first few holes had been carried out, tests were made for straightness and verticality, and at the first few holes there was every indication of a high rate of straightness and verticality being achieved.

The method of testing was very simple: a small electric battery torch, suspended on a piece of string, was lowered down the hole, and so long as the light could be seen in the 2½-in. hole, to all intents and purposes it was straight. If they could reflect the movement of the piece of string at the top of the hole by the movement of the torch, as indicated by the light moving across the hole relatively the same way, there was an indication of free suspension; and this in turn, the depth of the torch at the time being known, provided the possibility of calculating the extent of the verticality.

Great care was taken, in starting the hole, to get it going truly vertically, and straightness was secured by precisely the same methods as at Steenbras, namely by a follower with a diameter ⅛-in. less than that of the bit, at the end of the drill rod.

Overall, it had been found that about 99% of the holes tested were straight, whilst many of the holes in which the lighted torch had been suspended 80–90 ft down had indicated that the torch was still freely suspended at that depth, giving an indication of verticality. In view of the consistently satisfactory results achieved in the early stages, the testing of one hole in 20 was laid down as routine. Almost a quarter of a million feet of 2½-in.-dia. hole had been drilled satisfactorily by percussion methods.

It might therefore be logically stated that although there had been difficulties in drilling by percussive methods at Steenbras, that had not been the case at Tansa, where a very large footage of drilling had been carried out.

Mr Herbert Addison noted that when the authorities had first considered the heightening of the dam, they had found that trunk mains and filtration plant were "fortunately" already in existence, which could take the full augmented supply of water. Was that due to good fortune or to good management?

He had had the privilege soon after the war to be shown some of the Cape Town undertakings and he had seen signs of great resourcefulness. As the Author had said, there was indeed a problem by 1950, but in the meantime measures had been taken, if not to augment the total amount of water collected in the catchment, at least to augment the total amount delivered under peak conditions into Cape Town. That was particularly

important, because at that time—in the early stages of the war—convoys to the East and Far East had been routed round the Cape and they had made heavy demands on water at Cape Town. An emergency solution had been found by installing a booster plant in the existing main from Steenbras to the Cape Town storage reservoir, by which temporary supplies could be put through the pipe at a rate a good deal higher than by gravity alone.

Very soon afterwards, when conditions permitted, a new main had been put in, parallel to the existing main, and the filtration plant had also been installed.

It would further increase the value of the Paper then if the Authors could give a short summary of the various measures taken between the first heightening of the Steenbras dam and the second heightening described in the Paper. Moreover, it would show an interesting parallel with conditions in England in somewhat similar conditions: for example in the Manchester Water Works, where the supply had been increased by successive stages of boosting until finally a new source had been brought into operation; that was the Haweswater source, which was comparable with the new South African reservoir now approaching completion mentioned by the Authors.

Mr C. B. Barlow (Chief Engineer's Department, Metropolitan Water Board) observed that the original dam had a capacity of 600 million gallons and with a maximum height of 62 ft. In 1917, raising the wall 40 ft. had increased the storage to nearly 6,000 million gallons. Another 6½ ft had now been added and it would be interesting to know the new capacity of the reservoir.

When the first dam had been built, as a matter of extreme urgency at the time, it had been positioned higher up the Valley than necessary in order to leave the most favourable position free for a larger dam in the future. Presumably the possible use of such a site had now been abandoned, since reference was made on p. 31 to the possibility of a further major raising of the dam. Exactly how was it proposed that that should be done? Would the width of the wall be increased? What further increase in capacity was possible? Mr Lloyd-Davies, in a Paper [7] presented to the Institution in 1923, stated that the ultimate capacity of the basin was about 6,000 million gallons, but that had already been exceeded.

Turning to the strength of the dam, Mr Barlow said that the original dam was arched in plan, except for the short end portions, but it had been designed as a gravity section and he understood that an additional factor of safety had been attained thereby in order to guard against abnormal floods. Presumably that had not been achieved when raising it in 1927. In that connexion, it was interesting to note from Fig. 1, Plate 1, that the arched section had not required any post-stressing. He observed that the concentration of the vertical cables inserted in the wall was maximum at the junction of the flank walls with the arched portion of the dam, where the cables terminated abruptly, and it would be interesting to know whether any cracking had occurred in those regions, where, it would appear, considerable shear stress must have been induced.

It was stated on p. 26 that the calculations for the stability of the dam had as their basis an uplift ranging from two-thirds hydrostatic pressure at the upstream face to zero at the downstream face. On p. 33, the figure of two-thirds appeared to have been reduced to 0·6. It was also stated that when the holes for the cables had been drilled, with a few exceptions they had been watertight until 10–30 ft into the rock. It therefore appeared that the uplift was much less than assumed, and it would be interesting to know whether, in the light of that discovery, the dam before the present raising would have been stable under the conditions of the synthetic flood envisaged.

Had any account been taken in the calculations of the compressive stresses due to the thrust from the arched section of the dam being transmitted to the flank walls? In the discussion in 1932 [8] Professor Snape had suggested that the dam could well have been calculated as an arched dam.

[7] D. E. Lloyd-Davies, "The Works for the Augmentation of the Supply of Water to the City of Cape Town, South Africa." Min. Proc. Instn Civ. Engrs, vol. 216 (1922–23, Part II), p. 338.

[8] See p. 51 of reference 1.

It was stated on p. 24 that "the storage, supply, and distribution of water in Cape Town are the responsibilities of the City Engineer's Department." Was there still an officer officially known as "Water Engineer"? Cape Town had a population of half-a-million comparable to that of Leeds or the South Essex Water Company, with a daily supply of more than 25 million gallons. Was there any particular reason why Cape Town had not yet fallen into line with the majority of the larger water undertakings, which were looked after by water engineers employed whole-time on water supply? Generally, only smaller undertakings were the responsibility of the Council Surveyor and Engineer, who was also responsible for many other aspects of local government. In that connexion, Mr Delwyn Davies, who was known to most present and who had written at length on the organization and management of water undertakings, had stated in a recent Paper that modern water supply dictated the need for a specialist operational force. No longer, Mr Davies had said, could they expect any engineer to be expert in water, sewerage, highways, and town planning.

Mr A. C. Buck (a Senior Assistant Engineer, Messrs Binnie, Deacon & Gourley, Consulting Engineers) said the Paper brought to mind a regrettable aspect of British civil engineering as it was today and had been for 50 years—the apparent inability to conceive, develop, and exploit new ideas of major importance. He was thinking, for example, of prestressed concrete as developed by French engineers. Nothing of comparable magnitude had been done in the United Kingdom in the past 50 years.

The Paper described another example of French virtuosity. Mr Coyne had not only conceived it; he had worked it out to a practical end and brought it to fruition. His courage and tenacity of purpose were to be admired even more than his originality, for it was a very considerable undertaking to start such a project for the first time. The heightening of the Steenbras dam was also a bold undertaking, but it had been very successful and had achieved its end in a peculiarly fitting manner.

Now that Engineers were becoming accustomed to the use of high stresses in hard-drawn steel wire, a major point which arose was that of corrosion. He thought the ½-in. cover in the cables was sufficient for steel wire and provided permanent protection; but all the boreholes were not straight. Had the Author any qualms about cables touching the sides of the holes? The concrete and the rock underneath were, of course, grouted, so perhaps it was a point of little significance.

Some failures had occurred when loading the cable because of insufficient depth of grout in the hole. Could not some device have been provided for directly measuring the level of the grout? Apparently the load on the cable had been measured by both jacks and the elongation of the cable. The jacks might have an error of about 5%. Which of the readings had been taken in preference? What tolerances had been considered for the thickness of the packing pieces? When the packing pieces were put on the pedestal irregularities had to be controlled within fairly close limits.

In view of the difficulties experienced with the drilling, what was the Author's present opinion of using larger cables and fewer holes?

What protection, if any, had been given to the cables during transport to the site to prevent corrosion?

Turning to the cable heads, Mr Buck said a full-scale test was always of particular interest. Could the Authors give some details of the reinforcement in the heads and how the heads had failed when tested to destruction?

Mr Coyne's first essay in cable-anchored dams had been 20–25 years ago, and in that case he had assumed that the load from the rock which anchored the end of the cable was in the form of an inverted cone with a half-angle of 45°. When turning to the pertinent paragraph on p. 34, Mr Buck had hoped to see that there had been some development meanwhile and that there had been a different approach, but apparently Mr Coyne had kept to his 45°.

Reference had been made to a dam in Scotland which had been designed from the beginning on the same cable-anchored dam principle. Mr Banks, who had designed that

dam, had performed a very interesting experiment. He had excavated a shaft 18 ft into the rock, placed four jacks in the bottom, concreted the hole, and pumped oil into the jacks, taking readings at intervals until the jacks had burst at a load of 4,400 tons; the rock had given no evidence of failure. That compared with about 460 tons or so on Mr Coyne's theory, so it appeared that in some instances, at all events, Mr Coyne's approach was extremely conservative.

There were other means of approaching the important problem of how deep to take the cable. There were two theoretical solutions in existence, one a two-dimensional solution by Malan and the other a three-dimensional solution by Mindlin. The equations were cumbersome and not easy to use. Rock was not like the theoretical material on which mathematicians based their assumptions; it did not possess the same elastic properties in all directions; it was fissured and often stratified. It had to be remembered that rock was precompressed by its own weight. The horizontal planes were in compression and presumably so were vertical planes. If things were arranged so that the cable load was not increased beyond a point at which the tensile stresses of the horizontal planes did not exceed the compressive stresses superimposed by its weight, the problem might be approached with more confidence. It had been applied to Mr Banks's example, and assuming a conveniently placed horizontal fissure at the centre of the cable it was estimated that a load of approximately 4,000 tons would just begin to open up the fissure. Beyond that it was reasonable to expect the rate of movement of the anchor to increase with the load. If Mr Banks's results were examined, it would be seen that that sort of thing did in fact happen. Mr Banks had plotted the curve of movement of the anchor against load; there were several kinks in it, but the biggest kink, which seemed significant, was a load of about 1,400 tons.

Before Mr Banks's figures could be applied to the more theoretical reasoning, a correction had to be made. In that case the jacks exerted a force downwards and a force upwards and the tensile stress at the anchor was doubled. That meant that Mr Banks's load had to be doubled, obtaining 2,800 tons as the load where fissures began to separate at the anchor. Some sort of rough agreement exists, therefore, with the calculated 4,000 tons. Those figures were for a free surface at the top of the rock and not for a foundation load caused by the presence of a dam.

To investigate the extreme lower limit of the load capacity of an anchor, consideration had been given to the replacement of rock by sand; that might be assumed as equivalent to a very bad case of fissured rock. For forces applied vertically upwards on the anchor there was no spread of the load in sand when it was loose; the anchor would pull straight out. Where the sand was thoroughly compacted, the angle of spread was approximately half of the angle of 45° in rock assumed by Mr. Coyne.

Mr J. M. L. Bogle asked two questions on points which had not been raised in the discussion. The first concerned over-flows. In Fig. 2(a), the Authors had shown a raising which apparently meant a vertical drop on the spillway of about 15 ft. In cases of a major raising, such as the 45 ft which the Chairman had mentioned, would not that introduce problems which would lead to undesirable conditions of impact and surge? Would it not be possible to raise the crest by cantilevering on the upstream side?

The second question concerned the tension when the reservoir was empty. Would any tension be developed on the downstream face and, if so, would not the extent of it be the limiting factor in raising? It seemed that while the dam was full the loading of the cables equalized and helped the general stress distribution. If it were emptied, obviously there would be a reverse effect with tension on the downstream face. What was the result of the Authors' investigation on that point?

Mr J. S. Shipway (an Assistant Civil Engineer, Maunsell, Posford & Pavry, Consulting Engineers) referred to the question of the centre part of the dam functioning as an arch, and to the statement made on p. 34: "This assumption was supported by observations in the inspection gallery, since construction joints which intersect the gallery are

completely closed and watertight in the central part of the dam, whereas those on either flank show appreciable opening and in some cases slight leakage."

Presumably if the central part of the dam was acting as an arch then the thrust would have to be transferred through the flanks and thence to the abutments. In that case the joints in the flanks should also be closed, and not showing "appreciable opening."

The position of the inspection gallery was not shown on the cross-section and might prove illuminating. If it was located towards the top of the structure and on the downstream side, then it might be that the upper parts of the flanks were tending to act as horizontal beams between the centre section and the abutments, causing some tension on the downstream face. That would cause the joints to open.

Alternatively, the thrust from the arch might be resisted by the immediately adjacent sections of the flanks, leaving the remainder free to contract. The rock levels shown on the elevation, Fig. 1, would seem to confirm that.

The Authors' reply to the discussion, together with the correspondence on the foregoing Paper will be printed in a later number of the Proceedings.—Sec.

Paper No. 6092

THE USE OF BLAST-FURNACE SLAG AS A CONCRETE AGGREGATE

by

*** Eric Francis Farrington, B.Sc.(Eng.), A.M.I.C.E.**

(*Ordered by the Council to be published with written discussion*)

SYNOPSIS

Iron-blast-furnace slag is a non-metallic product developed simultaneously with iron in the furnace. At an iron and steel works the slag is plentiful and, following processing, can be used as a concrete aggregate.

The Paper describes the use made of slag as a concrete aggregate at the works of the Appleby-Frodingham Steel Company. The slag arises from the Company's own blast furnaces and conforms to B.S. 1047 : 1952. It is not suggested that all slags are suitable. Nevertheless, the experience advanced is sufficient to suggest that any iron-blast-furnace slag complying with B.S. 1047 : 1952 may be suitable as a coarse aggregate for concrete making.

How slag aggregate is produced

Molten slag from blast furnaces producing basic iron is tipped into shallow pits to cool and harden. It is then dug up and moved to the crushing plant for processing to B.S. specification for use as concrete aggregate, road stone, coated macadam, and railway ballast.

Formerly slag was hauled to the top and then tipped down slag banks. This process was wasteful in effort and the resulting slag lacked consistency. Bank slag is accordingly not used for making concrete aggregates at Appleby-Frodingham, but it is suitable for hard filling, or as a base course for roads. Several hundred thousand tons were used on the east coast of Lincolnshire in repairs to sea defences following the 1953 floods. It is also in regular use for the maintenance of training works on the banks of the rivers Humber and Trent.

Upwards of 20,000 tons per week of Appleby-Frodingham blast-furnace slag is now processed and disposed of for many commercial uses.

Early examples of slag-aggregate concretes

Foundations at Appleby-Frodingham bear mainly upon the remains of the Frodingham ironstone bed. This is a hard ferruginous limestone and presents few foundation problems.

Demolition of obsolete plant has shown that blast-furnace slag has been used as a coarse aggregate in concrete work at least since the beginning of the twentieth

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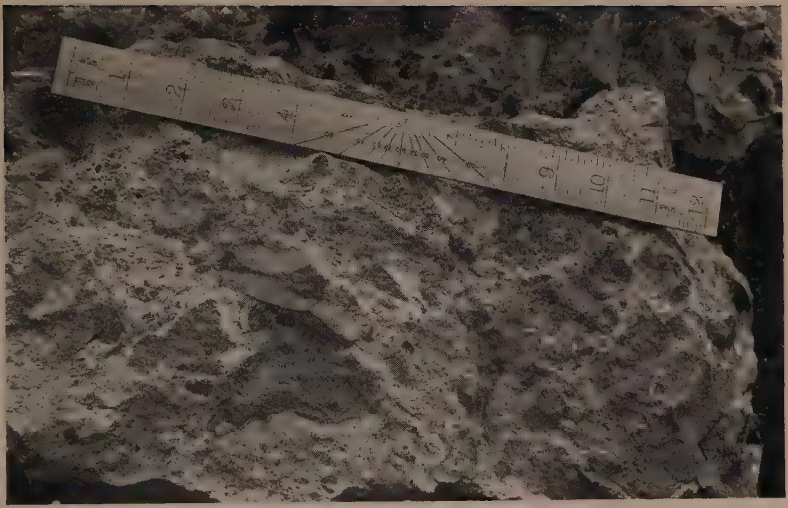

FIG. 1.—A CLOSE VIEW OF A SLAG-AGGREGATE CONCRETE POURED BEFORE 1910. THERE IS VARIATION IN THE SLAG BUT THE CONCRETE IS STILL SOUND

FIG. 2.—A CLOSE VIEW OF A STEEL JOIST IN A CONCRETE JACK-ARCH FLOOR. THE COMPARISON BETWEEN THE ATMOSPHERIC CORROSION ON THE UNDERSIDE OF THE FLANGE AND THE SURFACE CONDITION OF THE WEB IN CONTACT WITH THE CONCRETING IS INTERESTING. DATE OF THIS CONCRETE IS BEFORE 1910

Fig. 3.—Seraphim Ironworks development scheme. Precast concrete beams and slabs on the operating floor of the primary crushing plant

Fig. 4. –Seraphim Ironworks development scheme. Foundations for the ore-preparation plant

century (Fig. 1). Most of the old foundations are built of mass concrete but have incidental embedded steelwork, such as stanchions, holding-down bolts, tie-bolts, and steel joists in suspended floors of the concrete jack-arch type. No written specifications exist for these early concretes.

It is clear from inspection of the old foundations that the same attention was not given to correct grading and gauging as is done today. The lump size used is large, and there is a noticeable absence of intermediate-sized aggregate in some examples. Poor grading and segregation in placing has resulted in porosity in many of the old foundations. However, none of these shortcomings seem to have affected the slag aggregate, or the serviceability of the foundations for their original purpose. In recent years it has been deemed advisable in some cases to grout up some of the old foundations under pressure, and some have been encased where loadings have increased. Such faults or weaknesses have not arisen from the use of slag as the aggregate in the concrete, but rather from the method of construction. Moreover, embedded mild steel has shown negligible corrosion when the quality of the concrete has been satisfactory in other respects (Fig. 2).

In the past, economic necessity must first have dictated the use of a local material obtainable for the mere cost of recovering it. It remains within living memory that the use of no other aggregate would be contemplated when slag heaps were near at hand. This is still the policy today, but with 50 years experience available and the knowledge that the slag conforms to the relatively modern B.S. 1047 : 1952, no one now questions the suitability of slag for many applications in plain or reinforced concrete construction.

More recently the foundations of the Company's Appleby melting shop and plate mills, the construction of which was started in 1917, provide examples of slag-aggregate concretes when materials and techniques were more similar to those in current use.

This plant is still very active, and there is always reconstruction work going on which provides opportunities for looking at some of the older foundations. The earlier foundations used a large lump size; grading is poor, segregation and porosity in evidence. In later foundations the lump size becomes smaller, the aggregate better graded, and the resulting concrete consequently denser and stronger. This part of the plant provides a number of good examples of the durability of the slag-aggregate concretes under heavy industrial conditions.

Much of this concrete has a characteristic bluish colour and a characteristic smell when broken into, but the embedded mild steel shows negligible corrosion indicating that, whatever causes the colour and the smell, the slag is not deleterious to the concrete or mild steel.

Modern uses at the works of the Appleby-Frodingham Steel Company

Crushed and graded blast-furnace slag is now specified for all types of plain and reinforced concrete construction. To indicate the extent to which the Appleby-Frodingham Co. now use slag as a concrete aggregate, the following examples are mentioned.

On a recent development scheme, about 90,000 cu. yd of slag-aggregate concrete was poured. The majority was reinforced.

The foundations for the blast furnaces consist of cylindrical concrete blocks, 50 ft in diameter and 25 ft deep carried off the local ironstone. The top 10 ft of each block is reinforced with circumferential steel to resist temperature stresses.

To save construction time on the ore-preparation plant, the operating floor of the primary crushing building was formed of 40-ft-long precast beams in pairs (triple under railway tracks), supporting precast slabs 6 ft long, the whole being covered by 6 in. of in-situ concrete. These reinforced beams and slabs were mass-produced by a neighbouring precast concrete manufacturer (Fig. 3).

The raw-materials building for the sinter plant is a large reinforced concrete structure. The building is 400 ft long and the bunkers are 40 ft deep. The walls of the bunkers are of the counterfort retaining type. On one side, there are frames to accommodate material-handling bunkers, conveyors, and electrical switchrooms. The building steelwork is carried on top of the counterforts. The concrete structure is built off the rock, and was later filled around externally to plant grade thus partially hiding the real magnitude of the work (Fig. 4).

Foundations for heavy industrial equipment must necessarily be large, and none of the foregoing examples are particularly unusual or original. Nevertheless, progress and development is always continuing and, however minor, is worth recording. On this construction, slag aggregates were used in thin precast roofing slabs for the boiler-house and pump-house roofs. They are protected by a proprietary waterproofing.

Special reference to the pumping of slag-aggregate concrete is justified since previous attempts on other constructions had not been very successful. The mix used was 1 : 2·3 : 3·6 and was determined as the result of laboratory experiments. The water/cement ratio varied according to the nature of the work and the distance to be pumped. A proprietary wetting agent was used for some of the heavier operations. About 4,500 cu. yd of this concrete was satisfactorily pumped 700 ft horizontally and 100 ft vertically. The success achieved showed that, given perseverance and proper control, slag-aggregate concrete can be satisfactorily pumped.

Some practical characteristics

Crushed blast-furnace slag has a coarse surface texture. In mixing, regard has to be paid to the wetness of the aggregate, because as a result of its large surface area it takes more water than a smooth-surfaced stone to wet it thoroughly. If a dry low-water/cement-ratio concrete is needed, then it is recommended that ballast heaps be sprayed with water to saturate, so far as possible, the aggregates before use, to produce uniformity in the concrete. Occasionally wetting agents are helpful. To aid placing in confined situations which frequently occur in industrial maintenance work, it is often advisable slightly to increase the proportion of sand to obtain a more workable mix. Otherwise standard techniques of workability and finish apply. In some respects, slag has special advantages.

In an iron and steel works, there are many places where heat cannot be entirely excluded. In such places slag has been found to be a suitable aggregate for concrete, showing little or no tendency to spall. Used with high-alumina cements it has some refractory qualities; with ordinary Portland cements it has good fire-resistant qualities.

Its coarse surface texture leads to mechanical toughness. Similarly, slag-aggregate concrete weathers well, and is particularly resistant to frost.

Such concrete has good non-slip characteristics for flooring. This is important in industrial premises where iron-clad footwear is normal. Special finishes have been applied without difficulty provided that the usual precautions are taken to ensure a good key. At Appleby-Frodingham, finishes such as granolithic are laid whenever

possible immediately following the base course by using two screed boards; when the floor has set, it is impossible to separate the two layers.

As a result of its coarse texture and generally irregular shape, slag-aggregate concrete is extremely tough. It is resistant to shock loading, showing little tendency to spall or chip, and none to shatter. For this reason, it is difficult to break out deliberately with pneumatic tools. It stands up well to the vibrating or pounding loads transmitted from heavy industrial plant, and seems little affected by oil and grease.

In active production plants, modifications are always in hand which entail adjustments to the shape and size of concrete foundations for new equipment. Whilst slag-aggregate concretes may be difficult to break out, they have the advantage that holes can be drilled into them cleanly and accurately without spalling or splitting.

With regard to corrosion, despite the characteristic colour and smell previously mentioned, embedded mild steel shows no tendency to corrode where the concrete is in other respects satisfactory.

Conclusions

The works of the Appleby-Frodingham Steel Company are built on slag-aggregate concrete foundations, and the Company's confidence in the material has developed over many years. This suggests that slag conforming to B.S. 1047 : 1952 would always be worth while considering on equal terms with other aggregates, which are not always subject to the same critical examination so often directed to slag.

At Appleby-Frodingham, the development of the use of slag aggregate into reinforced concrete work indicates the reliance which the Company are now prepared to place upon it. Results are not confined to the short-term crushing strengths of test cubes or to laboratory experiments, but are evidenced by the plant itself, which is built on slag-aggregate concrete.

Acknowledgements

The Author is indebted to his colleagues for their help and constructive criticism, and for permission to publish this Paper he thanks the Directors of the Appleby-Frodingham Steel Company and the Appleby Slag Company, and the Directors of The United Steel Companies Ltd.

The Paper, which was received on the 24th May 1955, is accompanied by thirteen photographs, from some of which the half-tone page plates have been prepared.

CORRESPONDENCE on this Paper should be forwarded to reach the Institution before the 15th May, 1956. Contributions should not exceed 1,200 words.—SEC.

CORRESPONDENCE

on a Paper published in Proceedings, Part I, May 1955

Paper No. 6034

"A rational approach to the design of deep plate girders"*

by

John McHardy Young, B.Sc., A.M.I.C.E., and
Robert Elkan Landau, B.Sc.(Eng.)

Correspondence

Mr R. Wolchuk, of New York, stated that the Paper was valuable in its practical approach and in bringing to attention the problem of failure by yielding under combined stress.

Referring to the Authors' formulae (13) and (14) for investigating stability and safety against yield it should be observed that they took into account only the effect of stresses σ_x due to bending and shear τ_{xy}. Thus it was being assumed, as was also the case in other procedures recommended in various codes, that the stresses σ_y perpendicular to σ_x either did not occur or were so small as not to affect significantly the safety of the web.

However, in certain cases, large σ_y stresses might act on girder webs, which then ought not to be disregarded.

In addition to local disturbances near the vertical web stiffeners, etc., such stresses might be induced by direct load on plate-girder flanges (railway ties, concrete slab resting directly on top flange) and radial forces arising from curvature of bottom flanges at haunches.

In the above cases the σ_y stress might substantially affect both the elastic stability and the safety against yielding of web.

An added compressive stress σ_y reduced the elastic stability of a panel loaded by σ_x and τ_{xy}; conversely, a tensile stress $-\sigma_x$ would increase the buckling safety.†

The simultaneous action of tension and compression in perpendicular directions also tended to increase the "comparative stress," σ_c equation (9 in the Paper) which was the measure of safety against yielding. In the case of $\sigma_x = -\sigma_y$ (equal and opposite stresses) the web would yield under $\sigma_x = 0{\cdot}58\ \sigma$ yield or, in other words, the yield strength of web would be cut by almost half, compared with strength in pure tension.

The above considerations indicated that it was essential either to avoid large stresses σ_y in webs of girders or to take them into account in design.

Stresses in webs arising from direct load on flange were as a rule not too significant in cases of highway slabs resting on girders, but might be considerable in the case of railway tie loading.‡

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* Proc. Instn Civ. Engrs, Part I, vol. 4, p. 299 (May 1955).

† Formulae for those loading cases were available in "*Taschenbuch für Bauingenieure*" (Handbook for Civil Engineers) by F. Schleicher, 2nd Edn, p. 1020, Berlin, 1955.

‡ In regard to buckling safety of girder webs under simultaneous action of bending and concentrated railway tie loads, see F. Hartmann, "*Stahlbrückenbau*," p. 216, Vienna, 1951.

Stresses from flange curvature were directly proportional to the total force in flange and to its curvature. In long-span girders, where bottom flanges of considerable areas were curved at haunches, the radial stresses σ_y might reach the order of main bending stresses σ_x.

In the recent case of a proposed 258–387–258-ft span silicon-steel plate-girder bridge at New Haven, Connecticut, designed by D. B. Steinman, Consulting Engineer, New York, for which Mr Wolchuk was in charge of superstructure design, the maximum gross flange area of one girder was 164·5 sq. in., subject to a total stress of about 3,100 kips. With such a force in the flange, any radius of curvature smaller than, say, 100 ft would have caused substantial radial stresses in the web. Therefore, a smooth curvature was used for the haunches with a minimum radius of 208 ft (except for a sharp curvature within the bearing width at the support, where the radial stresses were taken directly by the bearing). The resulting radial stress on the web of about 1·2 kip per linear inch (compression) was deemed not serious.

Thus, careful geometric shaping of girders in order to minimize radial stresses σ_y appeared to be an important design consideration. In that connexion, long smooth parabolic haunches using large radii of curvature seemed to be best suited. Contrariwise, the quite commonly used "fishbelly" type haunches, requiring reversal of curvature and relatively small radii, appeared to be less appropriate.

It should also be observed that parabolic haunches, with the angle of slope gradually increasing towards the support, had the additional advantage of considerably reducing shear to be carried by the web, thus favourably affecting both the yielding and the buckling safety.

The Authors' suggestion to use high-strength steel for flanges and mild steel for webs of girders might be considered only if overstressing the portion of web between and near the flange angles was allowed (that was, if stresses in the extreme fibres of webs were still safely below the yield point) and, moreover, if web design was governed by its elastic stability.

However, in cases of deep continuous plate girders the web size might as often be determined by yielding safety. In such girders the bending stress in the web at the tensile flange was always rather close to the allowable stress in tension and the "comparative stress," due to tension σ_x, compression σ_y, if any, and shear combined might easily exceed the allowable value. Whilst elastic stability of thin webs could be easily and inexpensively increased by proper arrangement of relatively light stiffeners, the only way to increase the safety against yielding of the web was to increase its thickness, or to decrease stresses.

In the case of the 258–387–258-ft girder, the yield criterion applied to the top panel governed the choice of web thickness (11/16 in.) over support (maximum depth 21 ft 5 in.), whilst the elastic stability of the web, stiffened by two horizontal stiffeners located in the compression zone and vertical stiffeners spaced at 5-ft-4-in. centres, would be satisfied even with a smaller thickness.

If, in such circumstances, the use of two types of steel was contemplated, a higher-quality steel would be appropriate for webs than for flanges. However, that might not be necessary, since with some high-strength steels higher stresses were allowed for small thicknesses of material, suitable for girder webs.

Another possible way to avoid undue thickening of webs would be to use side plates at critical locations, thus reducing both shear and normal stress σ_y at the toes of flange angles.

Professor C. Massonnet, of the University of Liège, stated that he had himself devoted considerable time to the problem of plate buckling, first in private researches [21, 22, 23, 25] and later in researches financially supported by the C.E.C.M.[24, 26–30] (Belgian Commission for the study of metal construction). He wished to draw attention to three other Papers [31–33] not cited by the Authors.

He did not want to reproduce all the main conclusions of the ten Papers he had devoted to plate buckling, since the majority of the conclusions agreed with those of the Paper.

He therefore concentrated on the main results which differed from the Authors' conclusions or threw more light on definite points.

General results

He had examined the problem of the tolerance necessary on the flatness of a test plate to obtain well-defined buckling loads. It was shown that an initially slightly curved plate under a definite load in its plane took on larger additional lateral deflexions than a perfectly straight one, so that the membrane stresses in the middle surface of the plate were also greater. The effect of those membrane stresses was to blurr the phenomenon of "bifurcation" of the equilibrium at the critical load, so that nothing could be seen experimentally when the ratio initial deflexion/thickness of plate exceeded, say, 0·1.

In the same Paper he had explained why the general vibratory method he had proposed in 1940 for determining buckling loads of elastic systems [34] (which had been found excellent for the experimental determination of critical loads of bars and structures) was not practicable for plates and shells.

It had recently been shown by Green and Southwell [33] that the buckling load of initially flat plates followed approximately, in the post-critical range, a law of the form:

$$p_{cr} = p^{\circ}_{cr} + kA^2$$

where p°_{cr} denoted the critical load for zero lateral deflexion.

A „ the amplitude of the lateral deflexion.

p_{cr} „ the critical load for that deflexion.

k „ a numerical constant depending on the problem studied.

Dimensioning of the web

The stabilizing effect of membrane stresses already shown explained why it was sufficient to choose very low factors of safety against plate buckling. In fact, Professor Massonnet had shown in 1948, by detailed non-destructive and collapse tests on a large welded steel girder (13 m span and 1 m height with thin web $161 < b/t < 250$) that the experimental buckling loads were nearly midway between the theoretical ones for simply supported and built-in edges respectively, and that the load capacity of the girder was 2 to 4 times above that corresponding to the Timoshenko theory. It was therefore considered sufficient to adopt a safety factor of 1·35 against shear buckling.

Moreover, the tests had shown that the ratio experimental-critical load/theoretical-critical load (calculated by the Timoshenko formulae) was 20% higher in bending than in shear, so that it was considered sufficient to take a factor of safety of 1·15 against buckling by pure bending.

Apparently, those safety factors were definitely lower than the 1·75 proposed by the Authors on pp. 316 and 318. But, as the Authors increased the Timoshenko critical stresses by taking into account the clamping effect of the horizontal edges of the web, the difference between the two conceptions did not seem to be large, and the ratio 1·15/1·35 of the safety factors adopted by Professor Massonnet seemed to correspond roughly to that adopted by the Authors.

However, he did not understand why the use of the multiplying factor of 1·50 (proposed by the Authors on p. 319) should act only when horizontal stiffeners were used.

In the Paper it was proposed to design the web of a plate girder by analytical formulae. In order to reduce the complexity of the practical calculations, Professor Massonnet and Mr Greisch had devoted great efforts in the preparation of a general chart for quickly determining the web-plate thickness of a plate girder and the distance apart of the vertical stiffeners, taking the danger of buckling into consideration.[24, 25, 29]

He considered that the chart was an important practical step; it made possible the direct reading, without any calculations, of any of the three following quantities, web thickness, panel width, or safety factor against buckling, when the other two were given. The chart incorporated the combination of effects of the σ and τ stresses, as well as the reduction of stability due to plasticity. It was suitable for the two usual constructional

steels, A37 and A52, and it might also be applied not only to plain web panels, but also when there was a horizontal rigid stiffener.

It assumed that the reduction of stability when the critical comparison stress $\sigma_{cr} = \sqrt{\sigma_{cr}^2 + 3\tau^2_{cr}}$ exceeded the proportional limit of the steel was the same as for a compressed column.

In the Paper the same problem was solved by tacitly assuming (pp. 317–319) that the plate-buckling phenomenon remained elastic up to the yield point. It seemed that that assumption strongly oversimplified the problem. The same assumption applied to columns would be equivalent to replacing the actual Engesser-Shanley-Euler curve $R_{cr} = f(\lambda)$ represented by the curve ACD in Fig. 15 by the contour ABCD. The critical stress would become $\sigma_{cr} = R_c$ for $C \leqslant \lambda \leqslant 92$, which meant an important loss of safety in that field.

The oversimplification had definite consequences on the design, as would be apparent by discussing a later numerical example of the Authors.

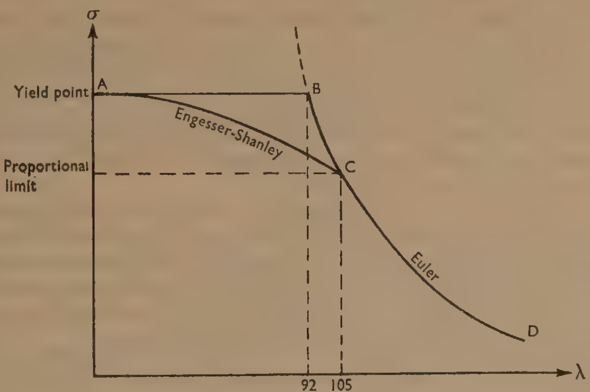

FIG. 15

Dimensioning of the stiffeners

Extensive non-destructive and collapse tests were made in 1951–53 on two large (13 m × 1 m) welded steel girders, the web of which had been reinforced by different combinations of vertical and horizontal stiffeners.[26, 27, 33] The slenderness ratios of the web panels (b/t = 250, 300, 350, and 425) were much in excess of the largest ratio permitted by most official regulations.

The main results of the tests were:—

(1) Generally speaking, the efficacy of a stiffener was restricted to a definite interval of load. That meant that the ratio deflexion of the stiffener/maximum deflexion of the panel outside the stiffener might well remain low so long as the load remained below a certain limit called the *limit of efficacy* and then would begin to grow abruptly when the load exceeded that limit.

The collapse tests showed that, to obtain a stiffener which remained straight up to collapse (that was, whose limit of efficacy nearly equalled the collapse load) it was necessary to give it a relative rigidity equal to k times the theoretical optimum rigidity γ. The values of k determined by the tests were:—

$$\text{Horizontal stiffener at } \begin{cases} h/2 - k = 3 \\ h/3 - k = 4 \\ h/4 - k = 6 \\ h/5 - k = 7 \end{cases}$$

$$\text{Vertical stiffeners} \qquad k = 3$$

(2) When such stiffeners were used, the *coefficient of utilization* U of the steel, defined by the ratio maximum stress in web or flange (calculated on the assumption of elasticity), divided by the yield point of the steel, reached values of 1·2, which were comparable to the values obtained recently in France on large rolled beams.

Professor Massonnet therefore considered that the dimensions of the stiffeners obtained by following the German regulations (DIN 4114) were definitely too low and on the dangerous side.

However, as Mr Shirley Smith pointed out at the Brussels Congress of Steel Information Centres in 1953, there was no compulsion to use stiffeners remaining straight up to the general failure of the girder. Mr Shirley Smith required only that they should remain straight up to the elastic limit of the girder. But the excellent value $U = 1{\cdot}2$ of the coefficient of utilization of steel obtained in Professor Massonnet's tests could not be guaranteed for such stiffeners. In fact, his first test girder, which was less stiff than the second one, gave a mean value of only $U = 1{\cdot}1$.

In regard to the design of stiffeners under combined bending and shear, Professor Massonnet wished to point out that, some years ago, he had made extensive numerical calculations to solve the case of a plate with a vertical stiffener at mid-distance, under (σ, τ) loading; the calculations had not been published. On their basis, Mr Greisch and he had proposed the following combination law. In a girder subjected to combined shear and bending, the necessary flexural rigidity of the horizontal and vertical stiffeners for separate stress conditions should be calculated and for each stiffener the greatest of the two values obtained should be adopted.

That law, of course, could not be said to have been generally demonstrated, because it found theoretical support only in one particular case. In any case it was more economical than the law proposed by the Authors on p. 327. Considering the safety factor of 2 that the Authors proposed to apply to the theoretical γ values on one side, and the k coefficients of 3 to 7 introduced by Professor Massonnet, on the other, the difference between the two design methods should not be considerable.

On the difficult question of interaction between stiffeners he wished to emphasize what he considered the very interesting idea on pp. 321–323 of the Paper, the fictive increasing of web thickness from t to T to take into account the fact that vertical strictly rigid stiffeners must be stronger when the web was horizontally stiffened than when it was not and so to be able to dimension the vertical stiffeners in composite stiffenings.

He felt that, in their Technical Note [30] Mr Greisch and he had failed to discover such a simple law, and the Note was imperfect in that respect. The proposal made by the Authors should therefore be incorporated in the Note.

On the other hand it was no advantage in all instances to assume the horizontal stiffeners to be supported by the vertical ones; in that respect the Authors' proposal should be widened in order to make possible the use of new theoretical results such as those of Scheer [32] which gave the necessary rigidity γ for crossed stiffeners of similar dimensions and for identical horizontal stiffeners at $h/2$ and $h/4$.

Another unsolved question should be emphasized—choosing the rigidities of two horizontal stiffeners acting simultaneously, as in the numerical example given by the Authors on pp. 327–332. In that respect they had assumed (p. 326) that it was permissible to make all horizontal stiffeners identical and of a section equal to that for the stiffener nearest the compression flange. In fact, another general law had been formulated [30] as follows.

In a panel reinforced by many stiffeners, the necessary flexural rigidity of the different stiffeners working together might be calculated by supposing that each of them worked alone on the *entire* panel.

Examples of that law were given symbolically in Fig. 16.

It should be stressed, however, that the only theoretical support that Mr Greisch and he had found for that law was given by the results of Scheer [32] already referred to. It should be stressed that new theoretical results pertaining to parallel horizontal stiffeners

or crossed stiffeners acting together were urgently needed. The corresponding theory was known; it was the energy method of Rayleigh–Rita–Timoshenko; the only difficulty was the complexity of the calculations, which should be done with the aid of an electronic digital computer.

Discussion of the numerical example given by the Authors

The value for the thickness of the equivalent web, given on p. 331, was definitely too low, because of the oversimplification made by the Authors referred to on p. 63.

In fact, when the chart [24] was applied to the same problem, it showed directly that the critical shear stress of the bottom panel was effectively the yield point; but for obtaining

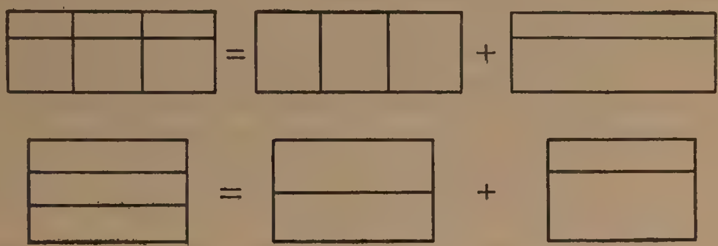

FIG. 16

the same critical stress in the equivalent web, a ratio $b/t = 108{\cdot}5$ is needed, which gave $T = 1{\cdot}43$ in. instead of 0·852 in. found by the Authors. On the other hand, when the curves drawn in the plastic domain between the proportional limit and the yield point were replaced by the corresponding elastic curves, as was assumed by the Authors, the chart gave $b/t = 188$, which yielded $T = 0{\cdot}824$ in. nearly the value found by the Authors.

Professor Massonnet could not agree with the opinion on p. 332 that one should adopt the curve $\delta = 0$ in Dubas's curves since the horizontal stiffeners were not made continuous. In fact, the stiffeners were attached to the web and compressed with it so that one employed the actual value of δ. Taking $\delta = 0$ meant an error on the dangerous side.

Future work

Research work was now under way in Belgium, under the sponsorship of the C.E.C.M. New tests were being prepared on

(1) Girders with single-sided stiffeners to determine the location under load of the exact position of the neutral axis of the stiffeners in bending and the width of the web strip participating in that bending.

(2) Girders with flanges and stiffeners of hollow closed section (welded angles) as proposed by Professor Dörnen (Fig. 17).

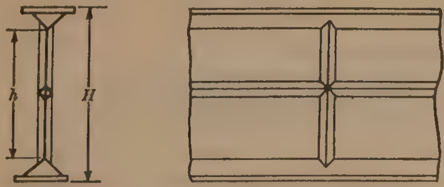

FIG. 17

The stability of the web of those girders should be improved by the reduction in height h and the clamping effect of the tubular flanges and stiffeners.

In order to have a theoretical basis for the design of such web panels, he had recently developed a general method of calculation of the buckling load of plates with simply supported or built-in edges, reinforced by stiffeners with torsional and flexural rigidities and subjected to shear and bending. He hoped to surmount the complicated numerical calculations with the aid of the electronic digital computing machine newly built in Antwerp.

Dr K. C. Rockey (Lecturer, Engineering Department, University College of Swansea) stated that one interesting feature was the way the Authors had dealt with the design of stiffened web plates. Before a correct procedure for the design of stiffened web plates could be developed, it was essential that the relations between the buckling stress of the stiffened web plates and the size and spacing of the intermediate stiffeners should be known. In dealing with the design of vertical intermediate stiffeners the Authors had referred to a recommendation made by Dr Rockey in 1952. Since making that recommendation he had completed an experimental investigation planned to determine the influence of the size and spacing of intermediate vertical stiffeners upon the buckling stress of the stiffened web plates. As a result of the investigation, which involved tests on several hundred different plate-stiffener combinations, it had been possible to derive new empirical relations which enabled the buckling stress of web plates, reinforced by uniformly spaced vertical stiffeners, to be calculated.

It was found that for any given value of the aspect ratio α_e, the K/γ relation consisted of two parts. At first, for values of γ up to a value termed the limiting value γ_L, a cubic relation existed between K and γ but that for a value of γ greater than γ_L, the value of the critical shear stress coefficient K remained constant at K_L.

Thus the new relation could be expressed as followed:

$$K = K_u + A(\gamma)^{\frac{1}{3}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which $$\gamma = \frac{EI}{Db}$$

where EI denoted flexural rigidity of stiffener.

Db ,, flexural rigidity of plate of width $b = \dfrac{Et^3b}{12(1-\mu^2)}$

K_u ,, critical shear stress coefficient of the unstiffened plate $= \dfrac{d_c{}^2t\,T_{cr}}{\pi^2 D}$

where T_{cr} was the corresponding critical shear stress.

K ,, critical shear stress of the stiffened plate, being equal to the maximum value K_L for values of γ equal to or greater than $\gamma_L \left(= \dfrac{EI_L}{Db}\right)$

A ,, a constant.

For double and single-sided stiffeners

$$K_L = 7{\cdot}0 + 5{\cdot}6\,(a_e)^{-2}$$

For double-sided stiffeners

$$\gamma_L = 27{\cdot}75\,(a_e)^{-2} - 7{\cdot}5$$

$$a_e = \frac{\text{clear web distance between stiffeners } bc}{\text{clear web depth } dc}$$

For single-sided stiffeners

$$\gamma_L = 21{\cdot}5\,(a_e)^{-2} - 7{\cdot}5$$

$$a_e = \frac{\text{stiffener spacing } b}{\text{clear web depth } dc}$$

From the above it would be noted that there was a difference in the behaviour of single nd double-sided stiffeners. For example, for a given stiffener spacing, a higher buckling tress could be obtained by using double-sided stiffeners in place of single-sided stiffeners. n addition, it would be noted that for a given stiffener spacing, the value of γ_L for double-ded stiffeners was greater than the value of γ_L for single-sided stiffeners. The expression r K_L quoted was obtained from the tests involving double-sided stiffeners and provided ightly lower values of K_L than those obtained from a similar expression derived from the ngle-sided stiffener tests.

One restriction on the use of the above formulae was that the relations were strictly alid only when the thickness of the attached stiffener leg was equal to or greater than e thickness of the web plate. It was now generally recognized that when the thickness the attached stiffener leg was less than the thickness of the web plate, the effectiveness the stiffener was reduced.

In addition to its function of increasing the buckling stress of the web plate, an inter-ediate stiffener had to operate as an effective member when the web plate was loaded eyond its buckling load. Tests had shown that intermediate stiffeners which had a

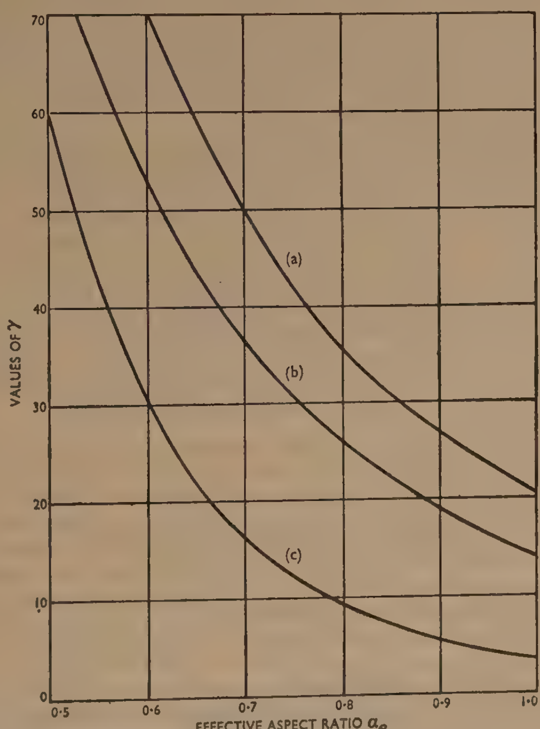

FIG. 18

Curve (a) recommended value for double-sided stiffeners

$$\gamma_L = 27{\cdot}75(\alpha_e)^{-2} - 7{\cdot}5$$

(b) for single-sided stiffeners

$$\gamma_L = 21{\cdot}5(\alpha_e)^{-2} - 7{\cdot}5$$

(c) value recommended by Young and Landau

flexural rigidity equal to EI_L would function satisfactorily when the web plate was loaded beyond its buckling load. It was therefore recommended that the intermediate stiffeners should possess a flexural rigidity at least equal to EI_L.

As shown in Fig. 18 the design value of γ proposed by the Authors was much smaller than the value recommended above, and in Dr Rockey's opinion, stiffeners designed in accordance with the Authors' recommendation would not be fully effective, especially if widely spaced. The stiffeners would not remain straight, but would buckle with the plate and consequently would not supply even a simple pin-joint support to the adjacent panels. That was important especially when both horizontal and vertical stiffeners were employed. Dr Rockey was therefore pleased that the Authors proposed to increase the flexural rigidity of the vertical intermediate stiffeners when they were used in conjunction with horizontal stiffeners. Although the procedure they recommended would result in the use of quite rigid vertical intermediate stiffeners it was clear that it would be desirable to obtain a more exact procedure based upon an experimental investigation.

The decision of the Authors to consider "the failing stresses for failure by buckling be taken as the theoretical instability stresses σ_{cr} and T_{cr} based on appropriate assumptions of edge-support conditions" must result in high load factors whenever the buckling stress was below the maximum permissible working stress. It was a pity that the Authors in that statement had associated the buckling of plates, which were supported along the edges, with failure, since it was now generally recognized that the buckling of such plates resulted merely in a redistribution of the stress system. In fact, as a result of recent research it was now possible to predict with reasonable accuracy the load at which initial yielding would occur in vertically stiffened webs.

With reference to web buckling due to bending stresses, the Authors quite rightly pointed out that such buckling was less important than shear buckling. That had been clearly established by tests conducted at Swansea on unstiffened web plates subjected to pure bending stresses. It had been found that the maximum stresses in the buckled web always occurred adjacent to the flange members and therefore since the flange stresses exceeded the plate stresses, the design of the flange members would ensure that at all times a satisfactory factor of safety against yield or failure would be obtained.

With reference to the fact that the German regulations did not limit the maximum slenderness ratio of webs, some restriction was desirable because if the ratio clear depth of web plate/thickness of web plate became too large, there was a danger, with webs subjected to bending stresses, of failure due to Grazier buckling occurring, especially if no vertical stiffeners were employed, or if they were widely spaced.

The Authors, in reply, observed that Mr Wolchuk had raised certain points which were not dealt with in the Paper, but which might have to be considered in certain cases. Considering first the case of "local" stresses which might occur in a railway underbridge where the sleepers rested directly upon the top flange or in a crane gantry girder, the Authors noted the references given by Mr Wolchuk, and wished to mention other papers dealing with that matter.[35, 36] His argument concerning the radial stresses occurring with curved flanges might be agreed, but when arched soffits were used with straight or very flat top flanges, the bottom flanges near the supports carried a considerable proportion of the total shear, which was usually neglected in estimating the shear to be carried by the web.

The Authors' suggestion to use high-tensile-steel flanges with mild-steel webs was purely tentative. In making that suggestion they considered that local yielding of the web might be regarded as similar to that due to the initial stresses caused by welding. It would, of course, be necessary to ensure that the yield point of the steel in the web was not reached under working loads. It was agreed that girders with curved flanges presented special problems which might rule out that suggestion. Mr Wolchuk had quoted the case of the $\frac{11}{16}$-in.-thick panel in the New Haven Bridge, which had been determined by yielding, and the Authors were generally in agreement with his remarks. The use of side plates was a possible alternative but they might cause complicated detailing.

Professor Massonnet's contribution was particularly valuable in view of the large

amount of research work he had carried out and the Authors were grateful to him for drawing their attention to the Papers he had quoted.

Dimensioning of the webs.—The 1·50 factor was intended to apply only to web panels that could be regarded as having one edge clamped; i.e., panels adjacent to the flanges of riveted girders. That factor was approximate only, but it did seem logical to the Authors that some difference should be made between panels which were partly clamped and those which were simply supported. The proposals of Professor Massonnet (like those of most authorities) did not differentiate between clamped and unclamped edges, resulting in a greater margin of safety in the case of clamped edges.

Buckling curves for plates.—The Authors were aware of the fact that their assumption of a sharp cut-off to those curves was debatable. Actually that had been taken as the constant upper limit of stress (irrespective of side-thickness ratio), which was the method used in the current B.S.449, the Code of Practice for Simply Supported Steel Bridges, also the recent draft revisions to B.S.153 and B.S.449. The method had the advantage of simplicity in use and might be justified by the fact that, as a rule, yielding of the plate as a whole occurred after any initial yielding had spread over the whole plate. The comparison with the buckling of columns appeared to be pessimistic, since it ignored the stabilizing effect of membrane stresses.

Dimensioning of the stiffeners.—The experimental results, presented in terms of factors to be applied to the theoretical minimum stiffnesses of horizontal and vertical stiffness, were of great interest. For vertical stiffeners the Authors had noticed that Professor Massonnet's proposed factor $k = 3$ gave results which agreed closely with the rules proposed by Dr Rockey.

Professor Massonnet had not stated, however, the effect on the buckling of a panel and

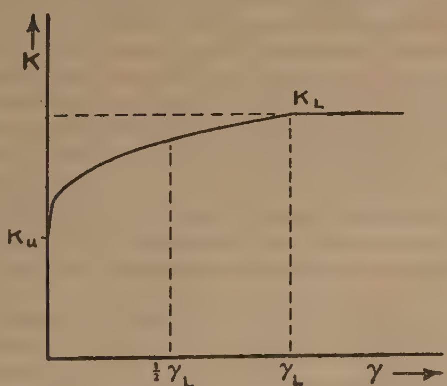

FIG. 19

on the "efficacy" of a girder of using a stiffener having a somewhat lower degree of stiffness. The Authors believed that that effect was not large. That could be shown (Fig. 19) by the form of the curve corresponding to the formula given by Dr Rockey.

$$K = K_u + A\gamma^{\frac{1}{3}}$$

Assuming $K_u = \frac{1}{2}K_L$ for the purposes of an example; then for

$$\gamma = \tfrac{1}{2}\gamma_L, \quad K \simeq 0{\cdot}9K_L$$

i.e., only 10% reduction in stiffener and plate buckling stress for a 50% reduction in stiffness.

The Authors noted that Professor Massonnet and Mr Greisch proposed to embody the method of the "substituted web" for horizontally stiffened panels in their Technical Note.

The question of interaction between horizontal and vertical stiffeners was complex and the method suggested by Professor Massonnet (Fig. 16) appeared to be a useful approximation. For stiffeners subject to combined shear and bending, the Authors felt that the rule proposed by Professor Massonnet and Mr Greisch was less conservative than their own proposals and that more theoretical and experimental evidence was necessary before it could be accepted.

Comparison of design rules.—The Authors had compared the stiffness required for vertical stiffeners (without horizontal stiffeners) by various authorities for a girder web ½ in. thick, 5 ft deep, with vertical stiffeners at 2-ft-6-in. spacing. The moment of inertia (in^4) was as follows:

Draft B.S.153 and Draft B.S.449	45·0
Rockey (double-sided)	36·7
— do —(single-sided)	26·9
Massonnet	30·9
Young and Landau	20·6
German DIN 4114	9·3

Those figures bore out Professor Massonnet's remarks regarding DIN 4114, with which the Authors were in agreement.

Design example—The Authors had stated that they did not consider the sharp cut-off to be undesirable but they believed that the effect of that approximation might be more marked in the case of vertical stiffeners designed by the "equivalent thickness" method. In such cases a somewhat higher factor of safety should be used, especially when the stresses in the web were close to the maximum allowable stresses. Professor Massonnet's criticism regarding the ratio of δ (Dubas curves) was accepted.

Dr Rockey's report on his research work and his proposals (which had already been referred to) were very welcome. Dr Rockey had stated that vertical stiffeners of flexural rigidity EI_L would function satisfactorily when the web plate was loaded beyond its buckling load and accordingly suggested that that reserve of strength should be taken into account. The Authors felt that that approach should be treated with some caution since (1) after buckling of web in shear the girder carried further load as a "tension field" beam which caused compressive stress in the stiffeners and additional compressive and local bending stresses in the flanges; and (2) if the design stresses were considerably beyond the buckling stresses, quite frequent buckling would occur. It might be appropriate to comment that the recent draft revisions to B.S.449 and B.S.133 contained proposals which would imply that stresses at failure (and under design conditions) might be well above the buckling stresses and that no special treatment of the stiffeners and flanges was necessary.

Dr Rockey's reference to Grazier buckling was somewhat obscure. In the opinion of the Authors that was unlikely to occur, since vertical stiffeners were almost invariably used and their pitch was unlikely to exceed 1½ times the clear depth of the web.

Conclusions.—The contributions of Professor Massonnet and Dr Rockey appeared to confirm that the buckling of plates did not occur in practice below the theoretical stresses and that there was a considerable margin of strength beyond the buckling load of a plate, provided that the plate was adequately supported. However, the required stiffness of the supporting members was found to be well above the theoretical values. Nevertheless it appeared to the Authors that the factors proposed by Professor Massonnet were on the high side for practical purposes and that rather lower values might result in only a slight loss of ultimate strength.

For design use, simple formulae were of course desirable, and the formula proposed for vertical stiffeners in the draft revisions to B.S.449 and B.S.153 had much to commend it. That simplicity, however, involved a safety factor varying with the ratio girder depth stiffener spacing and greater consistency might be obtained by the use of the curves in Fig. 18.

In the Paper, stiffeners designed according to various rules had been compared and that comparison had been further developed (pp. 320 *et seq.*). The divergence was so striking

that the Authors felt that the only satisfactory solution to the problem lay in some form of international specification.

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ELECTION OF MEMBERS, ASSOCIATE MEMBERS, AND ASSOCIATE

The Council at their meetings on the 18th October and the 15th November, 1955, in accordance with By-law 14, declared that the undermentioned had been duly elected:

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FOY, THOMAS ARTHUR WYNESS, C.S.I., C.I.E., B.Sc. (*Birmingham*).
GRANT, LEWIS COLIN.
KIER, OLAF, B.Sc. (*Copenhagen*).
SIM, ALEXANDER.

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APPLETON, SAMUEL ALAN, B.Eng. (*Liverpool*), Stud.I.C.E.
ARMSTRONG, MICHAEL CRUWYS, Stud. I.C.E.
ATKIN, DONALD MOSLEY, B.Eng. (*Sheffield*), Stud.I.C.E.
BARKER, MICHAEL CROSBY, B.Sc.(Eng.) (*London*), Grad.I.C.E.
BRADDOCK, RONALD EDWARD, Grad.I.C.E.
BROCK, GEOFFREY CLARENCE, Ph.D., B.Sc. (*Birmingham*).
BRYERS, THOMAS, Grad.I.C.E.
CLEMENTS, GERALD WILLIAM, B.Sc. (*Birmingham*).
CORNISH, JAMES WILLIAM, Stud.I.C.E.
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DALE, MICHAEL JOHN, B.Sc.(Eng.) (*London*).
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FELLOWS, ALWYN GRAHAM, B.E. (*New Zealand*).
GRADY, JACK, Grad.I.C.E.
GREEN, PETER BRIAN, B.Eng. (*Sheffield*), Grad.I.C.E.
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HODGSON, JOHN MATHEWS, B.Sc. (*Durham*).
HOWARD, KENNETH RICHARD, B.Sc. (*Bristol*).
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LEE, JOHN ANTHONY BRIAN, Grad.I.C.E.
LEYLAND, ROBERT BRETTON, B.Sc. (*Manchester*), Grad.I.C.E.
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McCAUSLAND, WILLIAM FRANK, B.E. (*New Zealand*).
MOLLOY, DOMINICK LEWIS, B.Sc.(Eng.) (*London*).
PARKES, EDWARD WALTER, Ph.D., M.A. (*Cantab.*).
PARVIN, ALAN HUSBAND, Grad.I.C.E.
POOL, DEREK CHARLES, Grad.I.C.E.
PRICKETT, ROBERT JAMES, B.E. (*New Zealand*), Grad.I.C.E.
RAVINE, ROBERT PETER, Grad.I.C.E.
SHUTTLEWORTH, PATRICK ESTILL, B.Sc. (*Witwatersrand*), Grad.I.C.E.
TARRANT, LUCIUS FREDERICK CHARLES, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
VENTER, JACOBUS JOHANNES, B.Sc. (*Witwatersrand*), Grad.I.C.E.
WELLER, NEVIL HORACE ERRINGTON, B.E. (*Queensland*).
WATTS, DERRYCK ALBERT, B.Sc. (*Bristol*), Stud.I.C.E.
WHITE, IVAN REGINALD, B.Sc.(Eng.) (*London*).
WILSON, GEORGE EDWARD BRIAN, M.A. (*Cantab.*), Grad.I.C.E.

Associate

CRITCHELL, PETER LYNE.

DEATHS

It is with deep regret that intimation of the death of the following members has been received.

Members

David Grant Anderson, B.Sc. (E. 1921, T. 1943).
Maurice George Bland, O.B.E. (E. 1910, T. 1932).
Robert Archibald Campbell, B.Sc. (E. 1912, T. 1946).
Eustace Herbert Cornelius (E. 1925).
Frederic William Duckham (E. 1903, T. 1911).
Eric John Lawson Gibson, B.A. (E. 1919, T. 1941).
Arthur Graham Glasgow (E. 1898, T. 1898).
Claude George Kent (E. 1909, T. 1933).
Robert Cowan Macdonald (E. 1933).
Ernest Minors, O.B.E., B.Sc.(Eng.) (E. 1907, T. 1946).
George Porteous Mitchell (E. 1932).
Stefan Georg Munz (E. 1948, T. 1950).
John Mawson, Rounthwaite, B.Sc. (E. 1909, T. 1939).
Frank Walter Shilstone (E. 1921, T. 1938).

Associate Members

William Eric Crosby Clark (E. 1944).
Ian Cecil Easton, B.Sc. (E. 1933).
Philip Cyril Egbert Fields-Clarke (E. 1920).
W.-Cdr. Leonard Nevill King, B.Sc.(Eng.) (E. 1936).
Thomas James McDonald (E. 1904).
Ernest Alfred Rex (E. 1924).
Albert Edward Valler (E. 1937).
Charles Marcus Allen Whitehouse (E. 1910).
Harben Robert Young (E. 1910).

OBITUARY

SIR ATHOL LANCELOT ANDERSON, K.C.B., who died on the 7th June, 1955, was born on the 17th January, 1875.

He was educated at Cheltenham College, and was subsequently articled to Sir James Lemon, M.I.C.E. He later served under the Civil Engineer-in-Chief, Admiralty, at Portsmouth, Malta, Rosyth, Hong Kong, Simonstown, Jamaica, Heligoland, and at Singapore. He was appointed Civil Engineer-in-Chief at the Admiralty in 1934, and retired in 1940. He was made a Knight Commander of the Order of the Bath in July 1939.

Sir Athol was a member of the Commission de Travaux of the International Commission of the Canal Maritime de Suez, and was Admiralty representative on Dover Harbour Board.

He was elected an Associate Member in 1900, and was transferred to the class of Members in 1913. He was elected Member of Council in 1936.

He is survived by two nieces and two nephews.

JOHN DEKEYNE ATKINSON, B.A., who died at his home on the 21st May, 1955, was born on the 2nd January, 1889.

Mr Atkinson, who was educated at Oundle and Cambridge, joined the Egyptian Irrigation Service in 1911, shortly after graduating. He transferred to the Sudan in 1915, and was henceforward concerned with a number of irrigation, pumping, and canal schemes. From 1929 to 1934 he was Director of the Hydraulics Section and of Hydraulic Research in the Ministry of Public Works, Egypt, engaged chiefly on advisory work relating to the Nile and the canal systems. From 1935 to 1939 Mr Atkinson held the post of Chief Engineer, Alexandria, and during the next two years was Chief Engineer, Iraq Irrigation Service. His last appointment was Director-General of Irrigation, Iraq, which he held from 1941 to 1946, when he retired.

He was author of the Handbook of Egyptian Irrigation (1934–35), co-author with A. D. D. Butcher of a Paper,[1] presented in 1932, on "The Causes and Prevention of Bed Erosion," and co-author with G. Cardiacos of a Paper[2] in 1945, "The Reconstruction of the Diyala Weir," for which he was awarded the Telford Premium. He was also a Member of the Société Royale de Géographie d'Egypte.

Mr Atkinson was elected an Associate Member in 1922, and was transferred to the class of Members in 1935.

ALFRED THOMAS BEST, who died at his home on the 16th May, 1955, was born on the 27th October, 1877.

He entered the family firm of Best & Sons, consulting drainage engineers in 1891, and from 1907 to 1910 was Deputy County Surveyor for Breconshire. In 1911 he became Designer and Assistant Engineer to the Port of London Authority, and in 1920 was appointed Chief Assistant Engineer of the British Petroleum Company. From 1925 onwards Mr Best was on the staff of Rendel, Palmer and Tritton, in the Harbour and Road Department. He became a Consultant of this firm in 1946.

His chief interest, on which he occasionally lectured, was in ports and docks, and

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[1] Min. Proc. Instn Civ. Engrs, vol. 235 (1932–33, Pt I), p. 175.

[2] J. Instn Civ. Engrs, vol. 25, p. 22 (Nov. 1945).

he was actively concerned in the improvements to the Royal Docks approaches, the Port of London improvements at Tilbury and West India Docks, Haifa Harbour and Jaffa port improvements, Chelsea Bridge and Waterloo Bridge, and many other structural works.

Mr Best was elected an Associate Member in 1904, and was transferred to the class of Members in 1931.

He is survived by his widow, one son, and two daughters.

LT-COL. JOHN ALDHELM RAIKES BROMAGE, C.I.E., who died on the 6th January, 1955, was born at Frome, Somerset, on the 7th May, 1891.

He was educated at the City of London School, and later at the City and Guilds Engineering School, where in 1912 he obtained their Diploma of Associateship in Civil and Mechanical Engineering.

After 18 months' practical training with Edwards and Company, Doncaster (under the personal supervision of the late Mr C. C. Johnson, A.M.I.C.E.), he joined, at the outbreak of the First World War, the 10th Battalion of the Middlesex Regiment, serving as a Transport Officer in India with the rank of Captain. From 1917 to 1919 he served in India, and after demobilization in 1920, he was appointed Assistant Sanitary Engineer by the Secretary of State for India, and was posted to serve in this position under the Punjab Government. The following year he was promoted to Executive Sanitary Engineer and later made Chief Engineer, Punjab. In 1941 he was created a Companion of the Order of the Indian Empire.

From 1920 onwards he was engaged on the design, construction, and maintenance of various water supply, sewerage, and drainage and allied works, amongst which can be named the Rawalpindi waterworks pumping scheme, Sargodha waterworks, Kurakshetra fair water supply and sanitary equipment, Thal water supply, and the sewerage project for the railway colony, Rawalpindi. He was also Superintending Engineer, Health Services, Delhi, and adviser on public health engineering to the Government of India.

Colonel Bromage was a recognized authority on water supply, sewerage, drainage, and sewage disposal, and in April 1940 presented a Paper [1] to the Institution on "The Sewage-Disposal of Delhi."

Since 1948, he had been acting as an engineering inspector for the Ministry of Health—later called the Ministry of Housing and Local Government.

He was elected an Associate Member in 1917, and was transferred to the class of Members in 1933.

In the second World War he joined the Royal Marine Engineers (Works) as a Major, and in 1944 was appointed Works Adviser to the Ministry of Works; later he was attached to Headquarters of the U.S. Army in London.

He is survived by his widow.

ROBERT ARCHIBALD CAMPBELL, B.Sc., who died at Christchurch, New Zealand, on the 7th September, 1955, at the age of 73, was born at Wanganui on the 17th October, 1881.

He was educated at Wanganui College, and, for a period of five years—1898–1902—at the University of New Zealand, from which he graduated in Mechanical Engineering. He began his career with the Grand Junction Gold Company, Waihi, and then spent two years, 1906–08, with Richardson Westgarth Ltd, West Hartlepool. Mr Campbell left Great Britain for India, where he served as a civil engineer

[1] J. Instn Civ. Engrs, vol. 14, p. 157 (Apr 1940).

on the construction of the Bengal and North-Western Railway, and in 1912 he joined the Public Works Department, Sydney.

During the first World War Mr Campbell served in Palestine as a lieutenant of engineers with the Australian Imperial Forces.

In 1920 Mr Campbell accepted a lectureship in the National School of Engineering at Canterbury University College, and in 1922 became the first Professor of Civil Engineering there. He resigned the professorship in 1930 to devote his time to private practice. In this capacity he was associated with many important engineering works throughout New Zealand.

Mr Campbell was elected an Associate Member in 1912, and was transferred to the class of Members in 1946.

He was a member of the Senate of the University of New Zealand and of the Council of the New Zealand Institution of Engineers, of which he was President during the year 1946–47.

He is survived by a widow and four daughters.

WILLIAM EAMES CATON, who died on the 18th February, 1955, was born on the 18th March, 1879.

He was first employed, in 1896, by the Luton Gas Company, in 1902 by the Midland Railway Company, and from 1905 to 1916 by the Derby Gas Company as Deputy Engineer and Manager. Mr Caton was then appointed, in 1916, General Manager and Engineer of the Oxford and District Gas Company. He became Managing Director and Chief Engineer in 1932, and in 1934 became Managing Director of the South Midland Gas Corporation and of the Banbury Gas Light and Coke Company. Before his retirement in 1944, Mr Caton had undertaken a number of important gas plant installations, notably in Oxford.

Mr Caton was elected to the class of Members in 1927.

He is survived by his widow and a son.

BRIGADIER WILFRID DINSEY CHAPMAN, D.Eng., M.C.E., who died at Melbourne on the 6th May, 1955, was born in 1892.

He was educated at Camberwell Grammar School and at the University of Melbourne, where he graduated B.C.E. in 1923, and gained the degree of M.C.E. two years later.

From 1915 to 1919 he served with the Australian Imperial Forces, at first in the Army Medical Corps, and later in the Machine Gun Corps.

He joined the Victorian Railways Construction Branch as a junior in 1908 and after graduation was responsible for the design of important railway structures, being a pioneer in the application of electric welding to structures. The use of this process in the reconditioning of an old road and railway bridge over the river Murray in 1924 is considered to be the first application of electric welding to such an undertaking.

In 1931, he joined the staff of the E.M.F. Electric Co. in order to develop and extend the field of electric welding amongst practising engineers generally. In 1936, he joined the staff of Australian Paper Manufacturers to investigate the use of eucalypts for making pulp and became a leading authority thereon.

On the outbreak of the second World War he left Malcolm Moore Ltd, to serve abroad with the Army Field Workshop in the Middle East, being mentioned in despatches.

Upon demobilization in 1945, he became Director of Civil Engineering, Railway

Standardization Division of the Commonwealth, and carried out extensive studies of uniform railway gauge in Australia. From 1944 until his death he was a Commissioner of the State Electricity Commission of Victoria.

He was a foundation member of The Institution of Engineers, Australia, and President in 1944; he represented this Institution on the Conference of Engineering Institutions of the British Commonwealth in South Africa in 1950.

Dr Chapman was elected an Associate Member in 1931, and was transferred to the class of Members in 1941; he was Hon. Secretary of the Victorian Committee of Management from 1936–40, and Chairman in 1943–44.

He is survived by his widow and a son.

JOHN HENRY WALES LAVERICK, D.Eng. (Sheffield), who died at his home in Sheffield on the 23rd March, 1955, was born on the 20th July, 1865.

He served his apprenticeship at Riddings Collieries, Derbyshire in 1881–4, and then studied engineering in the Mining Department at the Yorkshire College, Leeds (now Leeds University) under Professor Arnold Lupton, M.I.C.E. In 1886 he became Surveyor and General Assistant at Riddings Collieries, and in 1889, Manager. From 1904 to 1906 he was General Manager at Newdigale Colliery, and in 1907 was appointed General Manager and ultimately Managing Director at Tinsley Park Colliery.

Mr Laverick retired from active colliery work in 1941, but continued his work as Chairman of the Renishaw Iron Co. Ltd, and Director of seventeen other companies, principally colliery, coking and by-product, and lime concerns.

He was elected to the class of Members in 1924. He was a Past-President of the Midland Institute of Mining Engineers.

He is survived by his sister.

GUSTAVE PAUL ROBERT MAGNEL, who died on the 5th July, 1955, was born in Belgium on the 15th September, 1889.

After taking his engineering degree at the University of Ghent in 1912, Mr Magnel joined the firm of D. G. Somerville and Company in London, where he was Assistant from 1914 to 1917, and Chief Engineer until 1919. From 1919 until 1927 he was Director of Studies in Engineering at Ghent University, and then was appointed Professor in the School of Engineering.

He was a member of the Académie Royale de Belgique, the American Concrete Institute, the American Society of Civil Engineers, and the Institution of Structural Engineers, and held a number of foreign awards and decorations.

Professor Magnel was the author of a number of standard works on concrete, "Pratique du Calcul du Béton Armé" (6 vols, 1946/9), "Cours de Stabilité des Constructions" (4 vols, 1948), "Le Béton Précontraint" ("Prestressed Concrete") (1948), and "Le Calcul de la Poutre Vierendeel" (5 vols, 1934), and in February 1949, he delivered a Lecture[1] to the Institution on "Applications of Prestressed Concrete in Belgium." Professor Magnel was a widely recognized authority on ferro-concrete, and evolved the system of prestressing that bears his name.

He was elected Member in 1950.

He is survived by his widow and two sons.

SIR FREDERICK LEIGHTON VICTOR MILLS, Bt, M.C., who died in Durban, South Africa, on the 21st April, 1955, was born in Northumberland on the 14th March, 1893.

Sir Victor was educated at Cheltenham College and at the Royal Military Academy,

[1] J. Instn Civ. Engrs, vol. 32, p. 161 (Apr. 1949).

Woolwich, where his engineering studies began, and where he was later awarded the Sword of Honour. In 1912, he was commissioned in the Royal Artillery; he served throughout the first World War, and was awarded the Military Cross and Belgian Croix de Guerre. In 1919 he commenced engineering training as a pupil of Mr A. T. Walmisley, and in 1922 entered the Colonial Civil Service as Assistant and Executive Engineer in the Public Works Department, Nigeria. He was responsible for the inception and planning of the Entebbe Aerodrome, two of the runways being completed during his term of office. He was also concerned with the planning and the early stages of construction of the new main highway from Kampala to Entebbe. To assist the work on these two projects he installed, in spite of great difficulties, the first Soil Mechanics Laboratory in East Africa, which was soon to give considerable service to the surrounding districts.

In 1934 he became Senior Executive Engineer, and in 1936 Assistant Director of Public Works, Nigeria, administrating a works programme that included three major bridge constructions and four main aerodromes. From 1938 to 1942, he was Director of Public Works, Sierra Leone, and was then appointed Director of Public Works, Uganda.

Sir Victor retired in 1948; he succeeded to his father's baronetcy in 1953.

He was elected an Associate Member in 1934, and was transferred to the class of Members in 1944.

He is survived by his widow, Lady Mills, and a son.

ERNEST MINORS, O.B.E., B.Sc., who died in August 1955, at the age of 73, was born on the 15th March, 1882.

He was educated at King Edward VI Grammar School, and Mason College, Birmingham, and was then articled to the Borough Engineer and Surveyor of Wolverhampton. He became successively Engineering Assistant to the City Engineer, Worcester, then Deputy Borough Surveyor and, from 1924, Borough Surveyor, and Water Engineer to the County Borough of Darlington. He held this appointment until retirement in March 1947.

During his career, he graduated in engineering at London University, and was elected, from membership, to the Council of the Institution of Municipal and County Engineers, eventually holding the office of President of that Institution from 1945–46. In recognition of his noteworthy service as a local government officer, he was awarded the O.B.E. in January 1946.

He was elected an Associate Member in 1907, and was transferred to the class of Member in 1946.

JOHN ORR, O.B.E., LL.D.(Hon.), B.Sc., who died in Johannesburg in May 1954, was born in Scotland in 1870.

He entered Glasgow University in 1887, graduating in mechanical and electrical engineering, and was also educated at Coalbridge Mining College, the Royal Technical College, Glasgow, and the Royal College of Science. After varied practical experience with engineering and ship-building firms, he went to South Africa in 1897 on the inception of the South African School of Mines. In the following year he became Professor of Mechanical and Electrical Engineering at Kimberley, and subsequently devoted his life to education. In 1903, he went to Johannesburg to take up the position of Professor of Mechanical Engineering at the Transvaal Technical Institute, which eventually became the University of the Witwatersrand. In 1925, he resigned his de Beers Professorship on being appointed Director of the

itwatersrand Technical College, of whose Council he was first President. He tired from his position as Director in 1945.

Professor Orr served on numerous public bodies. He was President, 1908–9, the South African Institution of Engineers, and was President from its inception 1909 of the South African Standards Institution. He was particularly active in e sphere of technical education. He was a foundation Member of the Controlling xecutive of the Associated Scientific and Technical Societies of South Africa, of hich he was President in 1937–38.

Professor Orr served in the South African War; in 1919 he was awarded the O.B.E. r services rendered to the Disabled Soldiers Board. In 1936 the Honorary Degree LL.D. was conferred on him by the University of the Witwatersrand.

Professor Orr contributed many Papers on technical education and engineering to rious societies and was a member of a number of South African and other pro-ssional institutions, including the Institution of Mechanical Engineers, the Institu-n of Electrical Engineers, the American Society of Civil Engineers, and The merican Society of Mechanical Engineers, and he was an Honorary Associate of e Institution of Structural Engineers.

He was elected an Associate Member in 1904 and was transferred to the class Members in 1910.

MAJOR-GENERAL SIR CLIVE SELWYN STEELE, K.B.E., D.S.O., M.C., C.E., who died at Melbourne on the 5th August, 1955, was born in 1892.

He was educated at Scotch College and at the University of Melbourne where he aduated in civil engineering. He enlisted in the Australian Imperial Force early the first World War, attained the rank of Major, and received the Military Cross 1918.

In 1923 he commenced practice as a consulting engineer in Melbourne, specializing the steel frameworks of large buildings; his practice extended throughout Australia d included New Zealand and Fiji. The scope of his work included hospitals, nks, factories, churches, and the like.

Between the two wars he maintained a keen interest in military affairs, holding veral commands; he did much to advance and develop the practice of engineering the general life of the community and gave much time to the progress of The stitution of Engineers, Australia, of which he was a foundation member.

On the outbreak of the second World War he was appointed to the rank of eutenant-Colonel, then Brigadier, and finally Chief Engineer of the Australian nperial Forces; he sailed for the Middle East in 1940.

The end of the war saw him a Major-General with a D.S.O.

In 1944 he was awarded the Kernot Medal for distinguished achievements in gineering by the University of Melbourne and in 1953 he was created Knight ommander of the Order of the British Empire.

He was Chairman of the Victorian Committee of Management of The Institution Engineers, Australia in 1938 and Hon. Secretary from 1930–36. In 1955 he was ade an Honorary Member of the Institution.

He joined the Board of Directors of the Australian Paper Manufacturers in 1946 d it was largely through his efforts that the Brown Coal deposits at Bacchus arsh were developed to supply fuel to the paper works. He was also a director of ommonwealth oil refineries and other concerns.

Sir Clive was elected an Associate Member in 1918, and was transferred to the ass of Members in 1927. He was a Member of Council at the time of his death.

He is survived by his widow, Lady Steele.

WILLIAM STORRIE, who died on the 31st May, 1955, was born in Scotland on the 8th October, 1883.

On completing his technical education and apprenticeship in Scotland, Mr Storrie went to Canada in 1909 as Resident Engineer on the construction of the Toronto water-purification plant. In 1913 he became Chief Engineer of the John verMehr Engineering Company; in 1919, the firm of consulting engineers, Gore, Nasmith and Storrie was formed, eventually to become Gore and Storrie Limited, of which Mr Storrie was President.

Mr Storrie was associated with the design and construction of waterworks and sewerage projects in many municipalities throughout Canada. He was Past-Chairman of the Canadian Section of the American Water Works Association and Past-President of the Canadian Institute of Sewage and Sanitation. He received the George Warren Fuller Award and the Kenneth Allen Award in 1944 and 1947 respectively, for services in the field of water supply and sewerage, and the Dexter Brackett Memorial Medal for his Paper on the Toronto waterworks extensions, published in the Journal of the New England Water Works Association.

Mr Storrie was elected an Associate Member in 1909, and was transferred to the class of Members in 1929.

JOHN SIGISMUND WILSON, F.C.G.I., Hon. A.R.I.B.A. who died in London on the 17th March, 1955, was born in Cairo on the 15th October, 1875.

He was educated at St Paul's School, and at the Central Technical College (now the City and Guilds (Engineering) College) from 1893 to 1896. In 1897 he joined the firm of Sir John Fowler and Sir Benjamin Baker, consulting engineers, designers of the Forth Bridge and the Aswan Dam. Mr Wilson, with Mr W. Gore, investigated the problems of heightening this dam by using indiarubber models and successfully overcame the theoretical objections. A subsequent Paper [1] presented by them to the Institution earned the Authors the award of the George Stephenson Gold Medal in 1908.

During the 1914–18 war, Mr Wilson evolved the Wilson and Dalby director for anti-aircraft guns, and in the second World War he was engaged on the design and installation of anti-flood doors in the London tube railway network. He formed a partnership with Mr H. C. Booth (the inventor of the vacuum cleaner) and Mr C. W. Pettit, being largely concerned with structural steelwork for railway bridges and with the Piccadilly Tube railway. Later, however, he practised on his own and then took Mr John Mason as his partner.

When, in 1923, Waterloo Bridge was found to be unsafe, Mr Wilson became prominently involved in the controversy over its demolition, and supported the view that Rennie's 1817 bridge should be underpinned, widened, and preserved. Although demolition was eventually commenced in 1934, Mr Wilson had come to be recognized as an authority on the preservation of old bridges and other structures; in recognition of his work he was elected an honorary Associate of the Royal Institute of British Architects.

He was an active member of the British Association for the Advancement of Science, a member of the Newcomen Society, and of the Royal Institution, and a Fellow of the City and Guilds Institute, whose association of past students, the Old Centralians, elected him as their President in 1933.

Mr Wilson was elected Associate Member in 1903, and was transferred to the class of Members in 1927.

He is survived by his widow, a son, and a daughter.

[1] "Stresses in Dams: an Experimental Investigation by Means of India-Rubber Models" Min. Proc. Instn Civ. Engrs, vol. 172 (1908), p. 107.

CORRIGENDA

Proceedings, Part I, November 1955.

(1) p. 798, line 9 } *for* mW *read* kW
(2) p. 803, line 14 }

(3) p. 828. Figs 28, 29, 30, 31, which should accompany Mr Cornfield's contribution to the Correspondence on Paper No. 5989, were omitted.

(4) p. 837. Fig. 16, which should accompany Mr Cornfield's contribution to the Correspondence on Paper No. 5990, was omitted.

The abovementioned Figures are printed on loose pages supplied with this copy of the Proceedings for insertion at pp. 828 and 837 of the Proceedings, Part I, November 1955.

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